The Town Lattice Truss: An Appropriate Bridge Technology for Developing Countries

by

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ABSTRACT

The Town lattice truss is proposed as an appropriate technology for the Tshumbe Diocese of the Democratic Republic of Congo. This proposal is made based on an understanding of rural transport and appropriate technology and an in-depth analysis of the details of the Town lattice truss.

The nature and importance of rural transport and accessibility are presented, and bridges are identified as a key component in rural transport development. The concept of appropriate technology is presented along with a framework consisting of required and desired characteristics of any appropriate technology, including bridges. Structural materials are compared for use in bridges in rural areas of developing countries and timber is selected as the appropriate choice for the Tshumbe Diocese. Three existing timber bridges systems for developing countries are analyzed and compared, and the Town lattice truss is proposed as an alternative to all three.

The Town lattice truss is presented and described in detail with reference to a study of forty existing bridges in the northeastern United States that was conducted as a part of this work. Appropriate characteristics of the truss are identified and used to compare the truss with other timber bridge systems. The wooden pegged connections and chord structure are identified as unique components of the Town lattice truss and are the subjects of further analysis. Equations are developed for strength prediction and stiffness estimation for the wooden pegged connections. The chord structure is analyzed for strength and stiffness, which are determined to be combinations of underlying component properties based on the chord termination pattern that is used. A comprehensive set of possible chord termination patterns is developed and the best patterns are proposed for use in design. Finally, truss moment capacity is determined as a function of chord strength and stiffness properties and a simple methodology is proposed for the design of new Town lattice truss bridges.

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Chapter 1 – Introduction

1.1 - Background

In late 2003, Bishop Nicholas Djomo Lola of the Tshumbe diocese of the Democratic Republic of Congo initiated a relationship with MIT through a connection between Jerry Stanton at Catholic Relief Services (CRS) and Amy Smith of MIT's Edgerton Center and Department of Mechanical Engineering. In an email sent February 16, 2004, Bishop Djomo listed four major priorities in improving the situation of the people of his Diocese. The primary concerns were:

1) Improved access to clean drinking water.
2) Electricity for refrigeration in hospitals and health care facilities to increase the useful life of vaccines for Polio, yellow fever, and tetanus, among others.
3) Printing capability for the diocese to increase the availability of texts and manuals in the schools.
4) Capacity to maintain, improve, and expand transport infrastructure, with small-scale bridges as a particular priority.

The first three of the four priorities fit into standard themes seen in development around the world: water and sanitation, health, and education. The fourth, however, was somewhat surprising. It is generally acknowledged that transportation infrastructure is a key factor in improving the accessibility of many fundamental services and a crucial component in increasing the economic possibilities of an area. Despite this, it is often ignored at the community level. Many people consider transport infrastructure to be solely under the purview of larger regional or national organizations and beyond the capacity of local development organizations.

For Bishop Djomo, improving the transportation infrastructure was of particular concern to facilitate the distribution of food, both for basic subsistence and economic activities. With this in mind, the focus for the Tshumbe Diocese was on the internal rural road system and the local distribution networks. The current state of the road network is not good, with many sections of road difficult to traverse and bridges to cross the many rivers and waterways missing or unreliable. Figure 1.1 shows an example of the road between Lodja and Tshumbe, the two largest settlements in the Tshumbe Dicoese.

Figure 1.1 - Road between Lodja and Tshumbe in the Democratic Republic of Congo (Kerknet 2008)

While many of the roads are in poor shape, the bridges are of a more critical concern. A road may be difficult to traverse in a vehicle, but a river crossing without a bridge can be essentially impassable for both vehicles and pedestrians. In many locations, makeshift bridges fabricated from branches have been built to allow for crossing, but these are unreliable and dangerous. With the help of CRS, the diocese has developed some ability to build simple timber beam bridges, and the goal is to increase the capacity, span, reliability, and longevity of future bridges.

1.2 - Overview

The major objective of this research is to propose and develop an appropriate bridge technology for the Tshumbe Diocese. The current beam bridges have a relatively limited span and must be supported mid-river if the crossing exceeds this span. These mid-river supports are a critical component for the continuing function of the bridge and are also the component most vulnerable to decay due to contact with the water of the river. Increasing the possible clear span would allow for the elimination of these mid-river supports, and should yield a longer functional life for the bridge.

The general concept of an appropriate bridge technology will be explored in Chapter 2. First, rural transport in developing countries is discussed to understand the role a rural road bridge is expected to play in the development of rural transport as a tool for general economic development. This is followed by a description of appropriate technology as a general concept and the development of a new framework for the assessment and development of potential appropriate technologies. This framework is used as a method to discuss the characteristics that make a bridge technology appropriate.

Having developed an understanding of what makes a bridge technology appropriate, Chapter 3 will explore what bridge systems might be appropriate for the Tshumbe Diocese, in particular. Structural materials are first compared and contrasted with respect to use in developing a country, and the specific situation of the Tshumbe Diocese is addressed and used as a basis to select timber as the most appropriate structural material. Based on this choice, several timber bridge systems that have been proposed for, or built in, developing countries are studied to assess their feasibility. Neither of the longer span systems are considered ideal, and it is decided to explore another structural system for potential appropriateness. The Town lattice truss is selected as a potentially appropriate bridge technology for use in timber-rich rural areas of developing countries.

In Chapter 4, the Town lattice truss is presented, including technical details and dimensions based on a study of 40 existing Town lattice truss bridges in the northeastern United States. The characteristics of the Town lattice truss that make it a potentially appropriate technology are presented and used as a method of comparison with other timber bridge systems. Finally, a simple structural assessment of existing town lattice truss bridges is performed to determine that further analysis is needed, since existing bridges do not offer reasonable design parameters for use in new construction.

In Chapter 5, the unique wooden pegged connections of the Town lattice truss are analyzed. Strength prediction equations and stiffness estimation equations are developed for use in the design of the trusses.

In Chapter 6, the behaviour of the chords is addressed, with a particular focus on the chord termination patterns, a unique aspect of the Town lattice truss that has never been previously studied. Procedures for the prediction of strength and stiffness are developed.

In Chapter 7, the chord properties are used within the context of a Town lattice truss cross-section to determine the moment capacity of the truss. An example analysis is performed to illustrate the steps, and a simple design methodology is proposed. Finally, a set of proposed designs for new covered bridge spans are created using the procedures developed throughout this work.

Finally, Chapter 8 offers a set of conclusions based on this research and proposes areas of future work within the fields of appropriate bridge technology and the Town lattice truss bridge.

1.3 - References

Kerknet. (2008). "Lonya lo Lonya." Retrieved Sept 3, 2009, from http://www.kerknet.be/bisdomgent/content.php?ID=5679&VV485ZE=1.

Chapter 2 – Appropriate Bridge Technology

In determining an appropriate bridge technology for the Tshumbe Diocese, it is necessary to understand what makes for an appropriate bridge technology in general. This is a topic that has never been adequately addressed. Many development practitioners view bridges as being at a scale that is beyond the scope of their work and exclusively under the purview of structural engineers, who are integral agents in their design and construction. Even in the developed world, bridges are primarily functional in nature and the majority of decisions on the materials and structural systems are left to the engineer. This is in contrast to other structures, such as buildings, where many other factors must be considered since buildings have a significant impact on the people that use them and the environment they create.

In the developing world, most products and technologies should be considered to be more like buildings, with more than just pure function governing the design. All aspects of their environment, including the social, economic, and natural components, should be considered more significantly than they might otherwise. A conventional engineering approach may overlook these factors in favour of a pure focus on function, which is problematic and should be mitigated.

In this chapter, a general discussion of what makes and appropriate bridge technology is presented. Background on rural transport development and appropriate technology is important to understand all of the implications of design decisions. These topics are included and discussed with a focus on how their implications can be incorporated into the assessment and development of an appropriate bridges technology.

Finally, bridges will be discussed in particular, with a focus on existing organizations that work to provide bridges in developing countries, and the areas that these organizations overlook. The needs of the Tshumbe Diocese are in rural road bridges, which is an area that, until now, has not been generally addressed.

2.1 - Rural Transport in Developing Countries

It is first important to understand the nature and objectives of rural transport in developing countries. Dennis summarizes the discussion as follows:

> "Travel and transport are the means by which people gain access to the facilities and services they need for everyday life. Travel and transport are therefore a means to an end, the **real** need is accessibility. Rural households need access to an increasing range of facilities and services as they develop economically and socially and without this access, development will be restricted.
>
> Travel and transport involve time, effort and cost. These are the measures of the level of access to facilities and if they are too high they constrain opportunities and potential for development. The aims of accessibility planning should therefore be to minimise the need for travel and transport and to make that which is essential as efficient and cost-effective as possible." (Dennis 1998 p. 17)

It is generally accepted that access is a necessary, though not sufficient, condition for rural development and economic growth, and basic access is even considered by some to be a basic human right, on the same level as basic health and basic education (Lebo and Schelling 2001). Basic access is defined by Lebo and Schelling as "the minimum level … required to sustain socioeconomic activity" and is identified as providing "reliable, all-season passability for the locally prevailing means of transport."

The provision of universal basic access can be considered as the primary goal in rural transport development, with the desired secondary goal of long-term economic growth and prosperity. In the following sections, varying aspects of rural transport and accessibility will be presented, including a discussion of the access needs of the rural population, the state of rural access, the components that enable access, and strategies for improvement.

2.1.1 - What is Being Accessed?

In rural areas of the developing world, travel is an integral part of many common activities. Dennis (1998) divides these into four categories of activities, as listed below.

1. Daily Subsistence Needs – comprised of those activities that are part of daily life and are directly required to live, including the gathering of water and firewood and domestic food production.

2. Development of Human Capital – comprised of those activities that are not generally directly necessary for day-to-day subsistence but have a significant impact on quality of life and potential for future improvement, including education, information gathering, and health

3. Economic Activities – comprised of those activities that provide or increase income, which, in many locations, will be directly related to the growing and distribution of food, including farming, harvesting, delivering to markets, and acquiring supplies.

4. Other Social Activities – comprised of all other activities that do not fit in the other categories, and may include both more essential activities, such as trips to church and visits to government offices, or more social activities, such as visits to friends, sports and leisure activities, or non-essential shopping.

All of these activities take time, and the time spent traveling is essentially wasted, reducing the available time for other activities. By necessity, subsistence activities will almost always take priority over others. If the total time, including travel time, needed to perform these subsistence activities is too great, it will effectively preclude all others. Unfortunately, it is these other activities that include many of the necessary elements for socioeconomic improvement and an escape from poverty. Thus, the reduction of travel time, especially for basic subsistence, is an important task that has the potential for significant impact.

2.1.2 - The Current State of Access

The specific state of access will differ from region to region and depend on any number of factors, including topography, nature and location of services, types of agriculture activities, and local culture and traditions. Dawson and Barwell (1993) present four studies of rural transport based around community household surveys. This is in contrast to many previous studies, which were based on roadside surveys and, as a result, thought to overvalue the effect of road infrastructure by speaking only to those who use it.

The four studies are a 1986 study of the Tanga Region of Tanzania, a 1986/87 study of the Makete Region of Tanzania, a 1986 study of Ghana, and a 1988 study of the Aurora Province of the Philippines. The studies consist of both structured discussions with village leaders about the community and the local transport characteristics as well as questionnaire interviews with a random sampling of around 10% of village households.

It must be recognized that the included surveys were conducted nearly 25 years ago. Thus, while it is very likely that the exact situation in each location has changed, it is certainly not clear that it will have improved. Regardless, these studies offer snapshots of rural transport in developing countries, and are still thought to be indicative of the current situation in many locations.

The results of the studies can be summarized through two major metrics that help define the transport burden of a household or community. The first is the time spent traveling. Table 2.1 shows the average time required for a household in each area to reach specific services or facilities. It should be noted that the frequency of these trips is not uniform. Some activities, such as gathering water and firewood, occur daily; some activities, such as traveling to the grinding mill or the market, occur regularly and frequently; and some activities, such as traveling to the hospital, occur only irregularly and infrequently. The frequency of an activity is generally reflected in the travel time for that activity, with more frequent activities having shorter travel times.

Table 2.1 - Average time required by households to reach selected facilities (Dawson and Barwell 1993)

Survey location	*Water*	*Firewood*	*Cultivated land*	*Dispensary*	*Hospital*	*Grinding mill*	*Market*
Tanga	31mins	44mins	N/A*	1hr 45mins	N/A	1hr 51mins	2hrs 37mins
Makete	23mins	1hr 38mins	1hr 5mins	1hr 36mins	5hrs 40mins	1hr 42mins	3hrs 18mins
Ghana	25mins	43mins	48mins	1hr 40mins	2hrs 38mins	28mins	2hrs 8mins
Aurora	5mins	27mins	11mins	25mins	1hr 54mins	21mins	2hrs 8mins

* Average figure not available. However 80 per cent of households have fields within a 30-minute walk.

It is clearly evident that there is an extremely large transport burden in terms of time in at least the first three locations, and also for some activities in the fourth. The reason for the decreased average travel time for many activities in Aurora is due to a much higher level of access to motorized vehicles. Coming from the developed world, it is difficult to truly understand and appreciate this level of transport burden. For example, as opposed to having clean running water in individual homes, an average household in Tanga must travel one-hour, round-trip, in order to access water, which may not even be clean. Thus, we can begin to understand the nature of the problem.

When the frequency of trips is considered, it begins to become apparent how much of a transport burden exists in these areas. Table 2.2 includes the average aggregate time spent traveling per household in each area. To compare with a more commonly understood value, the hours per annum can be converted to hours per week by dividing by 52 weeks per annum. This yields 40, 48, 93, and 14 hours per week for Tanga, Makete, Ghana, and Aurora, respectively. Thus, it is the equivalent of what would be called a full-time job just spent traveling in Tanga and Makete. In Ghana, it is the equivalent of two full-time jobs and in Aurora it is somewhat less than a half-time job. This is most striking when it is remembered that travel is not a productive activity in and of itself. It is only a derived need that allows for other productive activities. It is clear from the numbers in Table 2.2 that there is a significant amount of productive time that could be recovered if the travel burden were reduced.

Table 2.2 - Some indicators of the scale of the average transport burden undertaken by sampled households (Dawson and Barwell 1993)

	No. of Trips/Annum	*Tonne-km/Annum*	*Time Spent/Annum*
Tanga*	1,799	84.6	2,083 hours
Makete	1,772	86.5	2,475 hours
Ghana	4,224	216	4,832 hours
Aurora	1,914	92	736 hours

* The Tanga data excludes crop marketing, and trips to markets and health facilities.

The second major metric derived from the studies is the magnitude of the load carrying activities, in tonne-km, which acts as a measure of the effort involved in traveling. Since much of the travel time in developing countries is used to carry goods, the load-carrying effort can be as significant as the time itself. Values for tonne-km per annum are given in Table 2.2. To give a sense of scale, 1 tonne-km is equivalent to transporting a 5 kg load 200 km, a not insignificant endeavour in itself when done on foot. For comparison, a typical 5 kg load in the developed world could be a moderately heavy backpack, containing a laptop and several books. A travel study in the United Kingdom (Office of National Statistics 2004) found an average distance walked per person per year of around 350 km for 20-29 year-olds. If all of this travel were done carrying a 5 kg load, similar to that described above, it would yield an average load carrying activity of 1750 kg-km, or 1.75 tonne-km, per person per annum. By contrast, an average household in the study areas has a load-carrying burden of more than 45 times this amount.

Figure 2.1 and Figure 2.2 show the division of time and effort, respectively, spent on transport for different activities for Makete. As can be seen, the vast majority of time and effort is spent on daily subsistence activities such as water and firewood gathering and crop production. The figures also subdivide activities in terms of gender. As can be seen, with the exception of crop production, the vast majority of the transport burden, both in terms of time and effort, is borne by females.

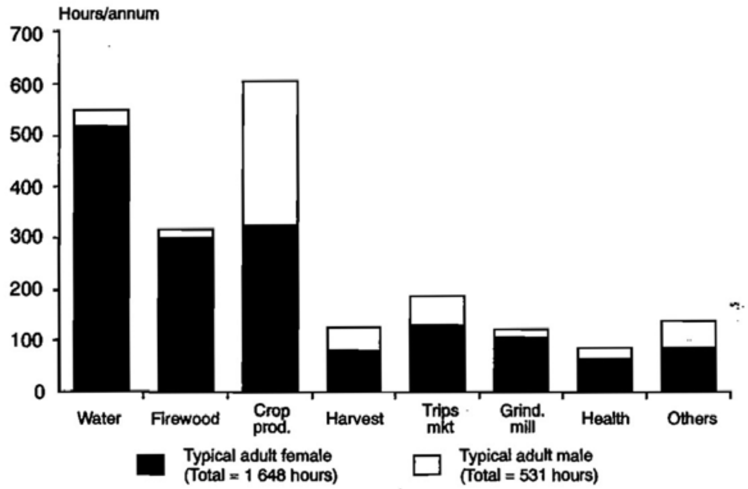

Figure 2.1 - Gender division of transport activities in Makete District (Dawson and Barwell 1993)

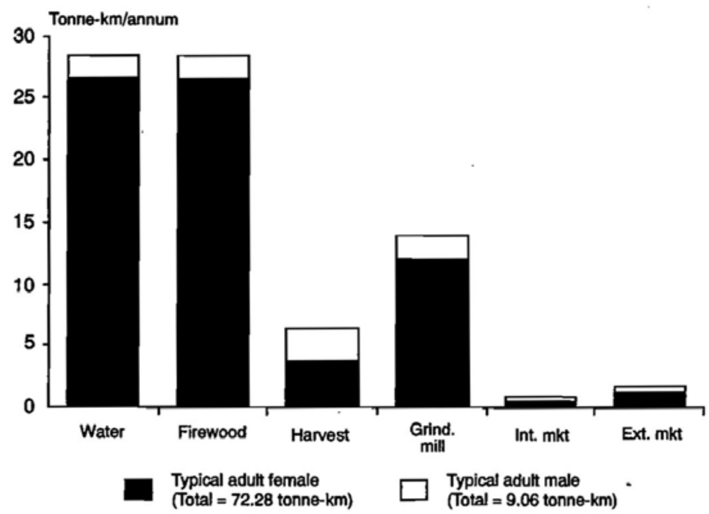

Figure 2.2 - Gender division of transport activities in Makete District (Dawson and Barwell 1993)

The main conclusion of Dawson and Barwell (1993) was that there is “a substantial transport burden to be undertaken by rural households, predominantly off the road network and on foot. In addition a body of complementary research not specifically focused on transport indicates that growing time constraints on rural households – and particularly on women in Africa – may be inhibiting their ability to increase agricultural output.” Since agriculture is the primary source of income for the rural poor, this indicates general limits for economic growth in rural areas.

In addition to observing the nature and scope the problem, they suggest how and why past transport development has not been effective. "Transport research and planning … have been largely focused on the needs of agricultural marketing and long-distance, motorized personal travel, with investment heavily weighted towards rural road infrastructure development. The surveys reveal that this approach does not address the totality of important rural transport needs: to a greater or lesser extent, tasks relating to the meeting of subsistence needs in the four study areas outweigh those relating to agricultural production and marketing; only a small fraction of the journeys made by rural people are outside the locality of their home village. Such journeys are rare in all study areas." Thus, they advocate a more balanced approach that extends beyond that highway-and-car approach and considers the needs of the rural poor.

Despite an element of bias towards trails and off-road networks (Barwell 2007), these infrastructure elements were not suggested as a replacement, but as a complement to rural roads. They write: "This is not, of course, to argue that rural roads are unimportant: motor vehicle access to the rural areas is essential for moving marketed crops; for the distribution of agricultural inputs such as fertilizer and seeds; and for the delivery of economic and social services. Motor vehicle access is also important in facilitating use of more centralized services such as hospitals; and in allowing longer-distance travel for social, educational, employment and business purposes."

Following from these conclusions, it is accepted that excessive transport burden is a problem in many areas of the developing world and needs to be improved. The first step in effecting improvement is to determine where, and to what extent, the transport burden is problem.

2.1.3 - Measuring Accessibility

In addition to transport burden, the rural development literature also frequently uses "accessibility" as a catchall term that expresses the level of access to services and facilities. Ali-Nejadfard and Edmonds (2000) write:

> "Accessibility is thus defined in terms of provision of access and the ease (expressed in spent time, effort and cost) with which a need can be satisfied."

Thus, accessibility is essentially the inverse of transport burden, with a high level of accessibility meaning a small transport burden, and a low level of accessibility meaning a large transport burden.

In order to make decisions about improving accessibility, it is helpful to have quantitative means to compare between alternatives. While these cannot completely replace qualitative assessment, they are useful as a broad first approximation assessment. These quantitative metrics can be developed from information gathered through household surveys, similar to those used in the studies presented above.

There are two simple quantitative methods found in the literature that can be used to assess accessibility. The first focuses on a specific facility or service and determines its accessibility to the surrounding population while the second focuses on a specific area and determines the accessibility of a variety of services to the people in that area.

The accessibility of a facility can be quantified by a simple “Accessibility Index” (Dennis 1998) of the form:

$$Accessibility\ Index = \frac{N}{T}$$

where:

N = number of people served by the facility or service

and T = average time of access to the facility or service.

The “Accessibility Index” can be a useful tool in planning when assessing between multiple destinations for moving an existing facility or providing a new facility. Each facility is considered to have a catchment area that includes all households that access that facility. The catchment area for each facility should be determined based on reasonable estimates of which facilities people will travel to, and areas should not overlap. Since the facilities being evaluated are considered essential, the total area covered by the catchment areas of the facilities must include all inhabited areas. The resulting catchment areas should be used for evaluating the “accessibility index” of all facilities by determining the appropriate population to associate with each facility.

Adding a new facility or moving an existing facility will require a reassessment of catchment areas and will typically have an effect on the indices of multiple facilities. The goal in any decision-making, then, should be to increase the index of as many facilities as possible while avoiding decreasing the index of any facilities.

The “accessibility index” will improve if more people are served by a facility at the same average travel time or if the average travel time is reduced for the same number of people served. In many cases, reducing the size of catchment areas, either by adding more facilities or changing the locations of existing facilities, will have the most effect through reducing the average travel time of the people served, even though it may simultaneously reduce the number of people served by each facility.

The second quantitative method looks at an area and evaluates the accessibility of different services and facilities for the people living in that area. For each sector, an “accessibility indicator” (Ali-Nejadfard and Edmonds 2000) can be calculated, using the equation:

$$AI = N \cdot (T - T_m) \cdot F$$

where:

N = number of households in the target area

T = average time spent to reach each facility or service

T_m = acceptable or target travel time for the specific facility or service

F = frequency of travel to the facility or service in a given period

The calculation of the Accessibility Indicator allows for comparison between the accessibility of different services from a given area or comparison of the accessibility of a given service between different areas. These comparisons can be useful in indicating services and areas that should have priority when planning accessibility improvements.

It should be noted that the "accessibility indicator" might be more appropriately named a "transport burden indicator" since a high value is indicative of poor accessibility, or a corresponding high transport burden, while a low value is indicative of good accessibility, or a corresponding low transport burden. It should further be noted that AI values can be negative, which is actually a desirable result indicating that the accessibility is better than the target level.

2.1.4 - Components of Access and Strategies for Improving Accessibility

As can be seen from the two accessibility metrics defined above, the major factor that determines accessibility is the time taken to access a facility or service. Decreasing the time of travel is the fundamental way to decrease the transport burden and improve accessibility.

There are two primary methods of decreasing the travel time for a specific service or facility. The first is to increase the proximity of the facilities or services that are being accessed; the second is to increase the mobility of the people accessing the facilities. Both have their advantages and limitations and should be considered together in any plan to improve accessibility.

The *proximity of facilities or services* can be improved by either moving existing facilities or creating new facilities. In many cases one or the other of these will not be an option for practical reasons. Water and firewood tend to be sourced from the natural surroundings, which cannot feasibly be moved; however, through long-term planning solutions, new sources, such as a well or a plantation, could be established. Some facilities that require constant supervision, such as hospitals, may not be feasible to establish in small locations. However, it may be possible to bring some of the services closer in the form of smaller health centers. Any change that reduces the need to travel will have a direct positive impact on the transport burden.

Mobility has two key components, the *mode of travel* and the *transportation infrastructure* that enables and facilitates the travel. The *mode of travel* can range from foot to motor vehicles, with various levels of intermediate modes of travel (IMTs) in between. IMTs can include wheelbarrows, carts, animals, bicycles, motorcycles, and motorized tricycles. These vehicles can reduce transport burden by increasing the load-carrying capacity of the individual, reducing the frequency of trips, or by increasing the speed of travel, or some combination of the three.

Transportation infrastructure includes all static physical components of the transportation network, and primarily refers to the trails, roads, and associated bridges of the network. Improvements to trails or roads can facilitate and increase the speed of travel at the current mode of travel, or can even enable the use of improved modes of travel. A smooth trail will be easier to walk on than an uneven trail, especially when carrying load. A trail that allows for foot traffic may still be too uneven or bumpy to allow for the efficient use of a bicycle or cart. Such a trail is furthermore unlikely to be able to support any sort of motorized vehicle.

It is important to note the interdependence of mode of travel and infrastructure in mobility. There is no benefit to introducing or improving the availability of IMTs if the transportation infrastructure is not sufficient to allow their use. At the same time, upgrading the infrastructure to allow for the use of IMTs will not necessarily have any effect if there is no increase in the availability of the IMTs. Thus, in order to successfully and significantly improve mobility, it is essential to consider both the mode of travel and the infrastructure that facilitates it.

The interdependence between mode of travel and infrastructure has not always been recognized, with the mode of travel more generally being overlooked. In the past, as a result of a variety of factors, transport development has largely focused on road building as a means of improving mobility. In many ways, the “highway-and-car” approach grew out of the colonial background of many developing countries. At the time, the primary objective in transport development was not to improve the lives of the residents, but to facilitate the journey of primary products to markets in Europe and North America. Later, free-market development strategies also favoured this approach, with foreign investors promoting the development of the transport network primarily for their own economic advantage. And finally, even when external economic motives no longer dominated the debate, many donors and development organizations still preferred a focus on road building since the output was visible and tangible, while taking little consideration for the efficacy or long-term sustainability.

Despite several decades worth of relatively large investment in transportation, Dawson and Barwell (1993) note that, as indicated by the four transport studies described earlier, a focus on road development alone resulted in very little improvement in the transport burden of the rural poor. To further underline the point, Dawson and Barwell describe a separate study in Malawi, which found that owing to a lack of vehicles in the country, the construction of an extensive road network had ‘facilitated travel’ but failed to ‘induce greater mobility’.

Any serious plan to improve rural accessibility should be a two-pronged approach that addresses both the proximity of facilities and the mobility of the people. Often, the first is overlooked within planning strategies, and the locations of facilities are taken as fixed. However, depending on the state of the rural transport system, it may be much less expensive, and more effective, to bring a service or facility to the people than to improve the network sufficiently to enable the people to reliably access the current facility. At the same time, it is entirely unrealistic to think that all services can be brought to all people. Even if a service or facility is made more local, there will always be parts of the population that still need to travel to it. Thus, the mobility of the people must also be addressed. Furthermore, bringing services to the people will only be feasible if the transport network is reliable and provides a certain minimum threshold access.

Within this research, the focus will be on the infrastructure component of the transport network, with a particular focus on the bridges, which serve a special role within the transport network. Though this work will have a narrower focus, it is recognized that there are limitations to merely upgrading the infrastructure and understood that such work must be considered within the context of a more general rural accessibility plan that supports an appropriate balance of approaches.

2.1.5 - Special Role of Bridges

Though bridges are often only a small component of transportation infrastructure when compared with roads and trails, they typically serve a critical function, enabling travel over otherwise impassable natural barriers such as rivers and gorges. Bridges are furthermore one of the more vulnerable components of the transport network, being intrinsically located near flowing water or highly variable topography. For both of these reasons, the design of bridges for rural transport networks is a topic that deserves attention. Since bridges are only a small component of the transport network, albeit an important one, there is a limited amount of work that has been done to develop appropriate systems and techniques, as compared to road-building, for example.

In terms of rural transport planning, bridges must largely be considered within the framework of transport infrastructure. Bridges are an integral part in many networks of roads or trails. In some cases, with deep gorges or rivers, a bridge may be essential to allow any crossing, without which a road or trail will be of no use at all. In these cases, a bridge can be an enabling component for a new link in the network, allowing connectivity where it was previously not possible. This can have a large effect on accessibility in reducing time of travel by simplifying circuitous routes around natural barriers into more direct links.

In other cases, the type and state of the bridge will be as important as the state of the trails or roads that it connects in determining the modes of travel that can be employed. Even if the trails that connect to either side of a gulley or waterway can support simple IMTs, such as carts or wheelbarrows, if they cannot be driven directly across and have to be unloaded and carried, they are unlikely to be used. Similarly, even if a bridge can support a given IMT, it will not enable the use of the IMT if it cannot easily be used on the trails that connect to the bridge. Taken to the extreme, the best-designed road bridge cannot enable motor vehicle travel if there are only trails or footpaths leading to it.

2.1.6 - Summary of rural transport in developing countries

Accessibility has been shown to be an important factor in many aspects of life in rural areas of developing countries. Improved accessibility has the potential to significantly reduce transport burden, freeing time and effort for many of the activities that are necessary in the reduction of poverty. Improvement to the rural transport network is a necessary component in improved accessibility, and is the primary focus of this work, although it is recognized that improved infrastructure is not sufficient to improve accessibility without addressing the other components of the rural transport system. This was shown in many projects where modern transport networks were constructed in developing countries without resulting in any improvement in the stake of the rural poor.

When working in developing countries, it is important that the system and its components be appropriate for the environment in which they are to be used. This is true for the design of the overall transport network, as well as the trails, roads, and bridges that make it up. This concept of 'appropriate technology' is discussed next, followed by a discussion of its implications for rural transport infrastructure.

2.2 - Appropriate Technology

Defining 'appropriate technology' is a relatively difficult task. People have been struggling with the semantics of it for many years and there are a resulting multitude of definitions which are influenced by the time in which they were created. They reflect the evolution of economic development theories, changes in attitude towards the developing world, and advances in underlying technologies, as well as differences in the knowledge and objectives of the definitions' authors. A summary of this issue is provided below, but first it may be helpful to understand the history of the concept of appropriate technology and how its evolution has related to economic and development theories of the last half-decade.

2.2.1 - History of the Appropriate Technology Movement

Most discussions of appropriate technology begin, justifiably, with E.F. Schumacher and his 1973 book "Small Is Beautiful: Economics As If People Mattered" (Schumacher 1973). Schumacher was among the first in the economic community to suggest that not all development is good and in fact, in many cases, too much development too quickly has serious negative impacts. As opposed to encouraging or imposing the sudden introduction of modern technologies, Schumacher encouraged a smoother economic evolution through the introduction of more appropriate intermediate technologies that would account for the specific social, cultural, geographical, and technological environment in which they would be used.

Though his seminal work was published in 1973, Schumacher's concept for intermediate technology was formally articulated in the early 1960s, and the underlying concept contrasted sharply with the dominant development theory of the time, in which technology played no significant role. Todaro and Smith (2009) explain that the first attempts to understand how nations develop economically were formulated based on the experience of rebuilding and modernizing many European countries that had been devastated by World War II. In this process, it was found that the rate of 'development' was closely associated with the rate of economic growth, which was itself almost exclusively dependent on the amount of money available to the country, either through savings or external investment.

At around the same time, former colonies in the "South" (a mid-20th-century term for the set of developing nations largely located in the southern hemisphere) were gaining independence and looking to shift from largely agriculture-based economies to modern industrial nations. Economists and development theorists believed the same patterns of development seen in Europe would be seen in these new developing nations and proposed models accordingly. Development was equated with economic growth, and all that was needed to spur economic growth was the investment of large amounts of capital.

At a superficial level, the first decade of independence for many of the former colonies seemed to validate this development theory. Large capital investments were provided to the developing world, and economic growth actually exceeded that of industrialized countries. But a deeper look at many of the countries showed that development did not necessarily follow economic growth. Modern sectors were established and productivity was improved in many areas, but the economic benefits of this increased productivity did not trickle down to the rural populations. In many cases, the rural population was actually further impoverished by the new technological investments, which often improved productivity at the expense of jobs.

This was the situation Schumacher encountered when advising in Burma and India in the late 1950s and early 1960s (Harrison 1983; Smillie 2000). Inspired by a variety of successful movements to embrace traditional technologies as a means of improving employment and productivity in rural areas, Schumacher posited that the development of alternative technologies that were more appropriate to the local situation would improve rural development by promoting local expertise and self-reliance. This improvement in the economic situation in the rural areas would in turn have a significant overall positive impact on the economic well-being of the nation as a whole.

Schumacher's renown as an economist lent weight to his concept, though many economic theorists still rejected the idea. Schumacher published several articles to which the public reception was generally favorable. Along with a number of sympathetic colleagues, Schumacher established the Intermediate Technology Development Group in 1965 and the Appropriate Technology (AT) movement was born. At first, largely due to a lack of funding, the development of appropriate technologies was limited. Much of the effort went towards trying to understand the problem, with the idea that appropriate solutions would present themselves, and, once they did, the follow up work to develop and implement the technologies would be relatively simple.

General acceptance of Schumacher's ideas within the economic establishment was slow in coming. Since early economic indicators from the 1960s implied that development was good in many of the nations of the South, there was no drive to question the conventional wisdom. This attitude was largely supported by the Pearson Report, released in 1969 and subsequently published under the title *Partners in Development*. This publication was the major product of the 1968 Pearson Commission, an international gathering of experts to asses the previous two decades of development assistance and propose its future trajectory (Pearson et al. 1969). The Pearson Report was perhaps overly optimistic as a result of the promising results of the preceding decade, and was largely focused on the need for increased capital investment to continue development. The report even advocated increased and facilitated investment by private corporations, not foreseeing the numerous problems this would engender (Smillie 2000).

In the early 1970s, it became more generally acknowledged that much of the economic growth in the South had not translated into overall national development and that the basic economic growth model was not adequate. The main failure of the linear growth model of development seems to have been correlating necessity with sufficiency. While increased capital investment and saving are necessary for accelerated economic growth, they are not sufficient. Many factors in the developing world, such as limited infrastructure, lack of education, and inefficient governance, limit the effectiveness of capital investment in a way that was not seen in the rebuilding and modernizing of Europe following World War II. New theories of economic development were developed to try to account for these additional factors (Todaro and Smith 2009).

This recognition of the failures of the existing methodologies led to reevaluation and restructuring by development agencies, resulting in a willingness to explore new ideas. This benefited the AT movement, and Schumacher and the ITDG found themselves with financing, allowing them to move beyond concept and begin searching for specific technologies that would satisfy the conditions of appropriateness. At the same time, the publishing of Schumacher's book in 1973 began to spread the message of appropriate technology to a much larger audience and technology choice began to be recognized as a serious consideration in development planning (Kaplinsky 1990).

While the decade was good for the AT movement, the world economy as a whole suffered significantly, with the impact being felt most strongly in the developing world. Two oil crises, a significant drop in the value of cash crops, and a continuing drop in development assistance all contributed to hurt the overall economic well-being of developing nations. Unemployment and poverty in their urban and rural areas continued to grow as a result of both this economic downturn and the many failed modernization schemes of the preceding decades.

> Many countries in the South, taking their cue from Northern economists, equated development with growth, and growth with industrialization. Massive investments were made in industry on the premise that these investments would form important linkages with other sectors of the economy. … For some, for a while, the strategy worked, but for many it did not. (Smillie 2000 p.10)

The Brandt Commission's report, published in 1979 was much less optimistic than the Pearson report, reflecting the many problems that had developed in the intervening decade (Brandt et al. 1980). Unlike the Pearson report and early economic development theory in general, the Brandt report acknowledged that technology plays a role in development. It recognized the many failures in large-scale development, focussing specifically on agriculture projects that had either not increased food production or led to the impoverishment of smaller farmers.

In the end, the Brandt report effectively endorsed the appropriate technology concept.

> The Brandt Commission pointed out that almost all advanced technology originated in industrial countries where it was developed for a different set of economic and production circumstances. Worse, the North accounted for 96 per cent of the world's spending on research and development, while much of the 'transfer' of technology to the South, of the 'choice' of technology, rested with

> Northern investors – multinational corporations – and with ‘experts’ involved in aid projects. (Smillie 2000 p.11)

Gaining respectability, popularity, and funding, the AT movement continued to focus on finding and developing specific technologies that it was hoped would have significant impacts on rural development. While there were some successes, practical results were limited by an exclusive focus on technology and hardware at the expense of dissemination and adoption. It was mistakenly assumed that if the perfect appropriate technology were found, everything else would take care of itself. This idea of one “perfect” technology is in many ways antithetical to the fundamental concept of appropriate technology, which is founded on the idea that how and where a technology is applied is as important as the technology itself.

By the mid 1980s, the AT movement began to move away from a purely technological focus and began to assess the reasons for the limited uptake of many of the appropriate technologies identified to date. The major lesson learned was that “the techniques of organization were as relevant to the transfer of technology as the technology itself.” (Smillie 2000 p.97)

In the 80s and 90s, a neoclassical counterrevolution in economic thought came to prominence, emphasizing free markets and the limited role of government (Todaro and Smith 2009). This was buoyed by conservative ruling parties in many of the industrialized countries, and resulted in a change in the nature and focus of funding for development assistance. Many aid agencies began to focus on small business development and entrepreneurship. For the AT movement, this largely meant the technology took a back seat to the context in which it was adopted – essentially the reverse of the early work.

While technology is important, much can be learned from a more business-oriented framework. Any production technology must be, or at least have the potential to be, competitive with existing techniques. If there is no possibility for equivalent or improved productivity or income generation, there is little chance of adoption, regardless of any other potentially desirable characteristics of the technology. In some cases, the potential competitiveness may be intrinsically linked with the adoption of related technologies, updated skills, or policy changes, which must be pursued concurrently with the technology in order to ensure viability.

It should be noted that a focus on the potential profitability of a production technology should only be of primary concern when considering value to the end user and not when considering value to the designer or supporting organization. An undue focus on profitability for the designer is likely to undervalue many of the other aspects of appropriateness. Additionally, it is important to consider that for many non-production-based technologies, a traditional economic assessment will be much more complicated if even relevant.

In addition to the primary importance of economic viability, a business-oriented framework also emphasizes the end user’s role in success. When developing a product to sell, it is important to consider what the consumer wants, needs, and is willing and able to use. Ignoring any one of these aspects will drastically decrease the likelihood that the product will sell. The same type of paradigm is true in appropriate technology, with the consumer being represented by the community or local population for whom the technology is proposed. Including the community and as many other stakeholders as possible in the decision-making process enables the designer to more fully understand the details of the end users' needs and wants. Considering and incorporating these preferences can greatly improve the chances of success in development projects. This type of participatory process is now frequently considered an essential component of development projects.

It should be noted here that not all end user ideas and behaviour are deep-seated beliefs, and some may be influenced through education about, and exposure to, a product or technology, though others will not. Regardless, it is essential to know the end user and their preferences to be able to attempt to influence them or incorporate them, as appropriate, in the product development.

By the late 1990s, it was acknowledged that a pure free-market approach was not sufficient to ensure development in many countries. Economic development theorists began to evaluate the conditions in which market forces were ineffective in fostering modern development and to propose new theories to account for these underdevelopment traps, including complementarities between different conditions and within inputs, coordination problems, and binding constraints on economic growth (Todaro and Smith 2009). In all of these cases, market forces alone are found to be insufficient to escape poverty or underdevelopment traps, and informed policy selection and government intervention are often necessary to overcome initial hindrances and allow entrepreneurial activities to succeed.

These current approaches to economic development theory are based on a synthesis of each of the early development theories, stressing the importance of a combination of entrepreneurial activity and policy intervention. This is the environment in which the modern AT movement must operate, and in many ways it has evolved accordingly. In addition to the technology itself needing to have economic value in the specific social environment, it is understood that a technology must also be transferred in a way that accounts for the many complicated interconnected market and policy forces that may inhibit its adoption.

A final recent trend is an explicit movement back towards the root of the movement, although tempered by lessons learned. Paul Polak is a proponent of this shift and writes "E.F. Schumacher was right on target by writing beautifully about smallness, even though he didn't focus enough on affordability and marketability" (Polak 2008, p. 76). The need for marketability has already been discussed, but affordability has not and is one of the cornerstones of this new trend. Polak espouses a focus on affordability, even proposing that in some cases "affordability is the most important consideration" (Polak 2008, p. 77).

A key principle of this trend is a shift in focus back to the individual or family and how they can effect change in their own lives. No matter how potentially effective a technology may be, it will make no difference if it is not cheap enough for the end user to purchase it in the first place. Innovations in microfinance and microlending have helped increase the range if what is affordable, but at the same time there needs to be concerted effort to reduce the costs of technology to what is reasonable for the intended customer.

2.2.2 - Defining Appropriate Technology

Any discussion of appropriate technology eventually comes to the point where the term must be defined. Unfortunately, this is more challenging than it might first seem. The difficulty arises in creating a single definition that manages to encompass all of the possible applications of appropriate technology while simultaneously not being so general that inappropriate technologies might also be potentially described. In fact, there is no specific definition of appropriate technology generally attributed to Schumacher, and he was impatient with those who devoted too much attention to words over ideas (Smillie 2000). Nevertheless, definitions do have their place, especially in situations when it is necessary to concisely describe the concept to those unfamiliar with it. Furthermore, it is difficult to evaluate or show that a technology is appropriate without a definition with which to compare.

When definitions of appropriate technology are created, they typically adopt one of two approaches to account for the large variability in what constitutes an appropriate technology. Willoughby (1990) designated these two common definition approaches as the ‘general-principles’ approach and the ‘specific-characteristics’ approach.

2.2.2.1 - General principles approach

The general-principles approach opts to create a traditional definition that is general enough to include many different types of appropriate technology while conveying the general essence of the appropriate technology concept. To illustrate the approach, four examples of general-principles definitions are presented below with discussion for each.

The first two definitions come from a single article from Andrew Conteh titled “What is appropriate technology?” The first takes the form of a traditional definition.

> “Appropriate technology is defined as any object, process, ideas, or practice that enhances human fulfillment through satisfaction of human needs.” (Conteh 2003 p.3)

This first definition is viewed as being far too general to be useful or arguably even accurate. Essentially, what is being said is that an appropriate technology must be useful, which is a valid statement. However, there is no more information provided about what makes one useful technology more appropriate than another useful technology. Following that logic, any technology that has a valuable function could be considered an appropriate technology. But, while that technology may be appropriate in some situations, it may not be in others, and this nuance is completely neglected.

Later in the article, Conteh adds a second definition in the form of a descriptive statement about appropriate technology which is much more apt.

> “The essence of appropriate technology is that the usefulness or value of a technology must be consolidated by the social, cultural, economic, and political milieu in which it is to be used.” (Conteh 2003 p.4)

While this does not take the form of a traditional definition, it invokes the critical nuance that was missing from the first definition, and is a much more telling and accurate description. A technology can be considered appropriate when it takes into account more than just the basic functional requirements, but also addresses the many other aspects that could have an effect on the usefulness of the technology. This description is one of the more concise and clear explanations of this concept.

The third definition comes from Willoughby, given as his own preferred definition as part of a prologue to a relatively detailed discussion about the many definitions of appropriate technology and the difficulties therein.

> “Appropriate Technology is … a technology tailored to fit the psychosocial and biophysical context prevailing in a particular location and period.” (Willoughby 1990 p.15)

This definition is reasonable in that it is general enough to be inclusive while also conveying the idea that context is an important factor in technology choice. That being said, it is not necessarily clear to a reader what is meant by the “psychosocial and biophysical context”. Lack of a clear knowledge of what the author intends these terms to mean introduces ambiguity, something generally avoided in a definition.

The fourth and final general principles definition comes from Harrison.

> “ ‘Appropriate Technology’ means simply any technology that makes the most economical use of a country’s natural resources and its relative proportions of capital, labour and skills, and that furthers national and social goals. Fostering AT means consciously encouraging the right choice of technology, not simply letting businessmen make the decision for you.” (Harrison 1983 p.140)

The ideas in this definition are sound, but the focus on ‘country’ and ‘national goals’ betrays a definite bias. While an overarching goal of national economic growth underlies rural development, the primary focus should be on the immediate context and not other indirect goals. Furthermore, the idea that there is an obvious ‘right choice of technology’ or that the primary driving force away from that choice is ‘businessmen’ seems overly simplistic, and may underestimate the many other challenges that face the rural poor for whom the technologies are intended.

All of these definitions are purposefully general and provide a broad umbrella under which most, if not all, appropriate technologies would fit. It is important for a general-principles definition not to be exclusive of potential appropriate technologies while also “emphasizing the achievement of a good fit between technology and its context” (Willoughby 1990). In this, Conteh’s second definition is viewed as being the most successful.

2.2.2.2 - Specific characteristics approach

Any general-principles definition without further description or explanation is inherently limited in its effectiveness in assessing or determining the appropriateness of a given technology. Without a greater specificity, the all-encompassing nature of the definitions means a variety of technologies could be formally argued to be appropriate, even while they violate the essential nature of appropriate technology, no matter how reasonably that nature is described. To resolve this deficiency, many opt to take the 'specific-characteristics' approach and define appropriate technology based on the qualities or characteristics it should or must have. Again, four example definitions using this approach are presented below with discussion.

The first definition comes from the introduction to the *Appropriate Technology Sourcebook*, where Darrow and Saxenian provide a list of characteristics that define appropriate technologies.

> "... tools and techniques that, in general 1) require only small amounts of capital; 2) emphasize the use of locally available materials, in order to lower costs and reduce supply problems; 3) are relatively labor-intensive but more productive than many traditional technologies; 4) are small enough in scale to be affordable to individual families or small groups of families; 5) can be understood, controlled and maintained by villagers whenever possible, without a high level of specific training; 6) can be produced in villages or small workshops; 7) suppose that people can and will work together to bring improvements to communities; 8) offer opportunities for local people to become involved in the modification and innovation process; 9) are flexible, can be adapted to different places and changing circumstances; 10) can be used in productive ways without doing harm to the environment." (Darrow and Saxenian 1986)

The inclusion of 'in general' at the beginning of the definition is crucial as it softens the dogma of this detailed list of features of appropriate technologies. Without this, the definition would be problematic since it is unlikely that any one technology could have all of the listed features. Certain characteristics, such as 'suppose people can and will work together' seem to be more about the process or state of mind than the specific tools and techniques, and it is unclear how this criteria would be applied or evaluated. Also, the final characteristic implying that there will be no 'harm to the environment' is unreasonable without defining what constitutes harm.

A list such as this is useful to provide many different examples of characteristics of appropriate technologies, which can supplement a general-principles definition. However, it has the disadvantage of not expressing any sort of weighting of importance or frequency between the points. Furthermore, it may not always be clear if such a list is intended to be comprehensive or only represents a subset of those qualities that may be related to appropriateness.

Jequier and Blanc provide the second specific-characteristics definition of appropriate technology at the beginning of a study on the growth of the AT movement worldwide.

It should be noted that this definition is not supplemented by any other discussion or definition.

> "Appropriate technology (AT) is now recognized as the generic term for a wide range of technologies characterized by any one or several of the following features: low investment cost per work-place, low capital investment per unit of output, organizational simplicity, high adaptability to a particular social or cultural environment, sparing use of natural resources, low cost of final product or high potential for employment." (Jequier and Blanc 1983 p.10)

This definition is primarily associated with production technologies, as can be read from the mention of workplaces, per unit output, and cost of final product. This is not a problem as long as this bias is recognized and the use of the definition is limited to production technologies and not applied in other realms of appropriate technology. It is not obvious that this is the case in the context from which the definition comes.

The reasoning behind the inclusion of a requirement for the 'sparing use of natural resources' is not understood and should not be considered a key characteristic of an appropriate technology. Obviously, resources should be used efficiently, and waste should be reduced as much as possible, but to limit the use of natural resources, regardless of how well they are used, is overly restrictive and potentially counterproductive.

Wade provides the third definition in a news-in-brief item in *Science* magazine concerning a composting toilet.

> "Appropriate technology differs from the other kind in being labor-intensive, accessible to its users, frugal of scarce resources, unintrusive on the natural ambience, and manageable by the individual or small group." (Wade 1980 p.40)

Intended as a short introduction to the concept of AT for a technical audience who may not be familiar with the term, the definition makes little attempt to be universal. It implies that all appropriate technologies must have the characteristics listed, though this is an unreasonable requirement. Though all of the features can be factors in the appropriateness of a technology, examples of appropriate technologies could be found that violate each. Even the composting toilet that is the subject of the article would be unlikely to be able to be considered labour-intensive.

Within the definition, two requirements deserve special note. First, 'unintrusive on the natural ambience' is a particularly broad requirement that seems as though it would be hard or impossible to achieve fully in most cases. Any sort of human activity could be described as being intrusive on the natural ambience, and appropriate technology is intrinsically linked to human activity. On the positive side, 'frugal use of scarce resources' is a reasonable statement of environmental conservancy, and is viewed as much more sensible and useful than the previous definition's 'sparing use of natural resources.'

Finally, Harrison supplements his general-principles definition, presented above, with a three-paragraph discussion of many of the specific characteristics that might be seen in an appropriate technology.

> "In the majority of developing countries the appropriate technology to use would look roughly similar. To soak up unemployment it has to create as many jobs as possible – hence it needs to be labour-intensive, using workers in preference to machines. It must be relatively cheap, because that enables the maximum number of jobs to be created with the limited funds available. But it must also improve income, so it needs to be more productive than traditional technology. And it must use scarce capital wisely, so it ought to produce as much output as possible for a given amount of investment.
>
> As skills are usually limited, machinery has to be simple to run and repair. It ought to use as much locally produced raw materials and equipment as possible, thus saving on foreign exchange and creating more jobs indirectly. It should be of a scale suited to the local market, otherwise its capacity will be chronically underused – the usual story with much of Third World industry. Further, it should contribute to the development of broad-based technological skill within each country. This means that, except in strategic cases, it should not be too far ahead of the local abilities to repair, cope and adapt, and should upgrade rather than destroy traditional skills. Environmentally, appropriate technology should be hygienic, conservational and non-polluting, using renewable sources of energy and raw materials whenever possible, with maximum reuse of industrial, animal, and human wastes and farm residues (e.g., for paper, fibre, fuel or building materials). It should satisfy basic needs and involve popular participation.
>
> ...But there is another practical set of requirements without which all of the other principles will remain pious hopes. Appropriate technology must be technically sound, economical to users and customers in comparison with the available alternatives, and socially acceptable in the light of the local culture and traditions. Most of the failures of appropriate technology out in the field can be traced to these the lack of one or more of last three down-to-earth prerequisites [sic]." (Harrison 1983 p.140-141)

This somewhat rambling list of characteristics suffers somewhat from a specific focus on machinery and workplaces, which was largely a product of the time in which this definition was conceived. Despite this, the characteristics are almost all generally applicable and tempered with a reasonable amount of qualification with respect to universal relevance. Additionally, the description specifically addresses three general requirements that supersede all of the other technology-specific characteristics. This clear distinction between general requirements and situation specific desirable characteristics is an important one that seems to rarely be addressed clearly.

Ultimately, there are many similarities between the specific characteristics definitions presented above. The specific characteristics proposed by Harrison seem to be the most universally applicable and also benefit from a clear hierarchy between requirements and desirables. This definition will be used as a starting point for developing a more universal characteristic list below.

Upon examination, none of the definitions that have been encountered in researching the topic have seemed to be complete, and many seem to lack a universality of application. Many of the definitions exhibit a bias towards the small and simple or a focus on workplaces, both of which can be traced back to Schumacher's original ideas. Unfortunately, this limits the applicability of these definitions for many modern applications of appropriate technology, the scope of which has expanded since Schumacher's time. Some technologies, including many of those related to infrastructure, will be inherently large and often not particularly simple. Many other technologies will be associated with daily tasks that are not themselves income-generating. In neither of these cases should the technologies be considered inherently inappropriate; it should simply mean that the frame of reference for evaluating appropriateness must be changed.

Specific-characteristics definitions have an advantage over the general-principles definitions in helping the reader to understand what types of technology might be considered appropriate, however they do not convey the general essence of the concept in as effective of a manner. Specific characteristics can be less easily misinterpreted to include inappropriate technologies, but are more likely to exclude potential appropriate technologies.

None of the above specific-characteristics definitions are thought to function as stand-alone definitions but could be used to supplement a general-principles definition. Similarly, a general-principles definition should also not be presented alone, but must be accompanied by a discussion or specific-characteristics definition that clarifies the intent. In either case, it is important to avoid or note field-specific bias so the reader can better understand the intent of the definition.

Willoughby (1990) concluded that the general-principles approach is superior and should be used in any overarching policy discussion, and the specific-characteristics approach should only be used in "specific contexts for which the circumstances have been clearly defined." However, this overlooks the disadvantages of the general-principles approach, which Willoughby himself notes can be "somewhat vague and amorphous". Ultimately, it is concluded here that a combination of the two is a necessity in any discussion of appropriate technology, and this approach will be adopted herein. The shortcomings of the specific-characteristics approach will be overcome by creating broader categories of characteristics and establishing a hierarchy of necessity between these different categories.

A further benefit of developing a robust universal specific characteristics definition for appropriate technology is that it may also be directly used as an evaluation tool. Kaplinsky (1990) explains that there are basically two approaches to the evaluation of appropriate technology, the social welfare approach and the specific characteristics approach. The social welfare approach is a quantitative method that attempts to provide a fully economics-based assessment of a technology that accounts for the unique aspects of developing countries through 'shadow prices' that adjust for distortions from straight economic assessment. This has the advantage of providing a clear numerical result for evaluation and comparison; however, there are many qualitative aspects that cannot be included, and there is even some question as to the accuracy of the quantitative 'shadow prices' that are included.

The specific characteristics approach is a more qualitative, multiple-criteria evaluation technique that includes less quantitative factors, such as environmental effects and social considerations that were excluded from the social welfare approach. Quantitative procedures such as economic assessment are still used where appropriate, but the final evaluation must compare between the many final considerations in a non-quantitative manner. In the end, the specific characteristics approach offers a more complete, if less deterministic, evaluation and is the primary method used by appropriate technology practitioners, while the social welfare approach can be used within the assessment to improve the economic portion of the analysis.

2.2.3 - A New Definition and Assessment Framework for Appropriate Technology

An attempt will be made here to establish a more universal functional definition through the specific characteristics approach that will useful in defining the appropriateness of a technology. This definition should be general enough to be inclusive of the vast majority of appropriate technologies yet specific enough to adequately convey the fundamental goals of appropriate technology.

Any assessment or reading of the specific characteristics mentioned below should be informed by an understanding of the basic concept of appropriate technology. One of the definitions of Andrew Conteh, presented above is considered a clear and concise conveyance of this concept, and is presented again below.

> "The essence of appropriate technology is that the usefulness or value of a technology must be consolidated by the social, cultural, economic, and political milieu in which it is to be used." (Conteh 2003 p.4)

All of the characteristics included in the specific-characteristics definitions above could be appropriate in a given situation, however none will be universally applicable. The characteristics can, however, all be related to a set of broader criteria, which themselves must always be addressed, even if all individual sub-requirements will not always be relevant. An attempt will be made here to establish a framework of criteria that covers all aspects of appropriate technology and within which all of the requirements noted above would fit.

Following on the example of Harrison, the specific criteria of appropriate technology will be divided into two categories with different levels of necessity. Firstly, there are a number of elements that are *required* for a solution to be both appropriate and worthwhile. These are:
- *a need and desire for the technology*,
- *functional adequacy*,
- *economic feasibility*,
- *sustainability*, and

- *no serious adverse environmental effects*.

The components of this list are similar to criteria that would be expected to govern a project in any context, however there are specific nuances when considering appropriate technology that will be described below.

Secondly, there are an additional number of elements that are *desirable* in many appropriate technologies, and largely differentiate these technologies from their conventional alternatives. These are:

- *positive environmental effects*,
- *local empowerment*,
- *use of local materials*, and
- *use of labour-based methods.*

However, while these criteria should be strived for and may help distinguish between different levels of appropriateness, they will not always be relevant or possible to achieve. Figure 2.3 shows a schematic of the required and desired characteristics of an appropriate technology.

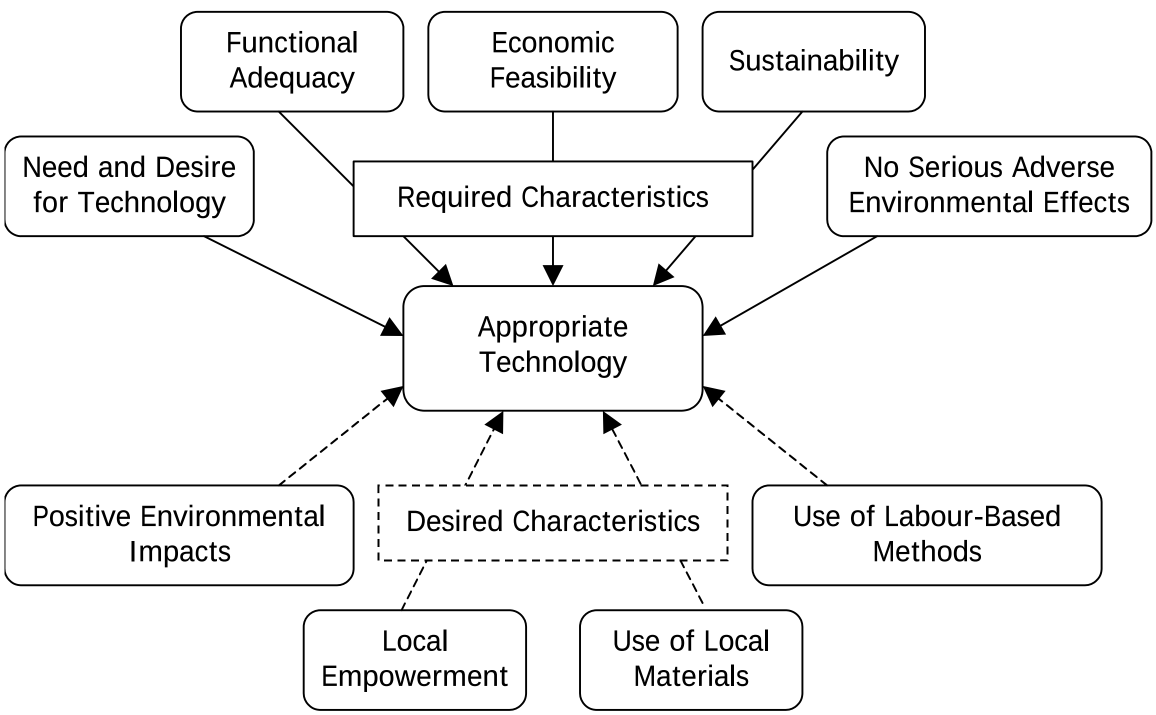

Figure 2.3 – Proposed assessment framework for an appropriate technology

This assessment framework is intended to be used as a starting point in the assessment of appropriate technology, and as framework from which to begin a discussion of what characteristics make a technology more or less appropriate. This is not intended to a quantitative multi-objective analysis, which may have a place in specific aspects of appropriate technology, but will be unfeasible to apply in an overarching assessment due to the qualitative nature of many of the characteristics of appropriate technology.

2.2.3.1 - Required Elements

2.2.3.1.1 - A Need and Desire for the Technology

To justify the development and introduction of a technology, it must serve a useful purpose. There must be an objective void to be filled, deficiency to be alleviated or eliminated, current technology or process to be improved, or other problem to be solved. The existence of one of these potential uses, however, is not enough. It is equally important that the local population views the problem as a problem, wants the problem solved, and, even more specifically, wants the problem solved in the method proposed.

The equal consideration of the perception of the local population is one of the qualities that distinguish appropriate technology from other, in that it incorporates the social and cultural environment in evaluating the technology's value. Perhaps most important is the recognition that simply because something is perceived to be a problem by some does not mean it is automatically considered to be so by others.

In some cases, a lack of knowledge or experience may be the driving factors in why a population would not understand why something could be considered problematic or how a technology might improve their current situation. In these cases, education about a topic or exposure to a technology may or may not change perceptions and opinions. While it is reasonable to attempt this type of conversion, it is important to recognize when it is not successful, and avoid imposing a solution on a resistant or unwilling population, as the failure of any such effort is inevitable.

2.2.3.1.2 - Functional Adequacy

In any technology, one of the primary criteria must be that it meet minimum functional requirements. If this cannot be achieved, the technology must be considered inadequate for its proposed use.

In an appropriate technology, one of the challenges is determining the minimum requirements in a reasonable way. An understatement is clearly unacceptable, but an overstatement can be equally problematic since it is important not to increase the size, complexity, or cost beyond what is necessary. This is often in contrast with traditional technology development, where a reasonable increase in cost and complexity can be accepted if it accompanies a reasonable increase in function, even if that functionality is simply a convenience or will only be used rarely.

It is obviously worth increasing function and value if the associated costs are insignificant, but this is not likely to be common, and in most situations the goal should be to meet or just exceed the minimum functional requirements. Thus, the challenge is to ensure that the minimum requirements are established appropriately, high enough that the problem is solved or that productivity is increased sufficiently to make the adoption of the technology worthwhile, yet not so high that the marginal benefits are outweighed by the marginal increases in cost.

Paul Polak addresses this topic as part of a discussion of how to make technologies affordable. One of his guidelines is to "make redundancy redundant" (Polak 2008, p. 77)) To explain, he writes: "If a western engineer is asked to design a bridge capable of holding a ten-ton load, he is likely to build it to hold a thirty-ton load to lower the risk of a lawsuit if the bridge collapses." This is obviously a vast oversimplification, but does highlight the tendency in the developed world to design to exceptionally high standards, sometimes adding capacity that will never be used. Quality and affordability are tradeoffs, and in the developing world, where affordability is of a much greater importance, it is often necessary to sacrifice a certain level of quality in order to make a technology that is appropriate.

2.2.3.1.3 - Economic Feasibility

With any technology, it is necessary to have the finances to pay for it. In appropriate technology there is an additional requirement to ensure that the cost is reasonable.

The reasonableness of the cost is primarily a matter of relative scale. If the capital investment is too large relative to typical local income, it may lead to a variety of other problems, including a drain of resources away from other projects, a decreased sense of ownership and responsibility in the user, and unreasonably expensive maintenance requirements.

It may at first seem unnecessary to evaluate the scale of the cost separately from the ability to pay for it. It could be argued that if there are enough funds available, then the cost is inherently reasonable, and similarly inherently unreasonable if there are not enough funds. In many cases this argument is valid, particularly if the eventual user is providing all of the funds. However, in developing countries, and particularly with the rural poor, development is typically fostered through the addition of external funds, usually in the form of aid or loans. This artificially increases the amount of money available beyond what the eventual user could reasonably afford. While this is necessary in rural development to break the poverty cycle which denies the rural poor any chance of advancement, it is important not to drive this artificial investment too far.

This aspect of appropriate technology can be attributed back to Schumacher, who was initially largely focused on reducing the cost per workplace such that it would be in reasonable proportion to the income of the worker. This attitude was in direct response to many unsuccessful large-scale development projects that were founded on the basis of the early capital-focused economic development theories. Large amounts of capital were invested in expensive modern technologies, which both benefited the rich more than the poor and ceased to be useful early in their design lives as a result of a lack of expertise and money to maintain them.

2.2.3.1.4 - Sustainability

Sustainability is often overused as a term and it is important to define how it is intended for any particular application. In its broadest terms, sustainability means the ability to maintain the current state of something over a given timeframe. One of the primary requirements for sustainability is the continued availability of all resources required for operation or maintenance.

In the developed world, the term sustainability most often refers to the environment or environmental-sustainability. Here the scope is typically broad, including all aspects of the global environment and its natural resources, and the timeframe is long, speaking on the order of generations or even to the indefinite future. While this scale of concern is worthwhile to consider and will have impact on humanity as a whole, it is in some ways a luxury that is only available to those for whom immediate survival and an escape from poverty are not primary concerns. Thus, while environmental concerns are important, they will be addressed separately and are not a consideration in discussions of the sustainability of an appropriate technology.

For the developing world, sustainability of a technology will be focused on its continuing function and useful operation. The timeframe in this case will be typically be on the order of years, up to the order of decades, depending on the technology. The resources to be considered will be those that have a more immediate impact on the technology, including *money*, *materials*, and *expertise*, and each will have somewhat different requirements for operations and for maintenance and repair.

Money here refers to the finances needed to cover all costs that occur after the initial capital investment. Operations costs, which include material inputs, energy, and wages, are likely to be relatively regular and predictable, though there is always the potential for unforeseen events or occurrences. Maintenance and repair costs are likely to include both a somewhat regular and predictable maintenance component and a completely unpredictable repair component.

While both the operations and the maintenance and repair costs should be included in an assessment of economic feasibility, it is usually the case that the maintenance and repair components are not. Sometimes this oversight is not purposeful, but in other cases it reflects the attitude of the initial donor of investor, who may believe that the users should be able to supply upkeep costs since they did not have to provide the initial capital. However, if the initial capital investment is too large, the maintenance and repair costs may be far too large to be shouldered by the users. This topic is addressed in the previous discussion of economic feasibility.

Materials here refers to both the material inputs and tools needed for the operation of the technology but also any materials and tools that might be needed for maintenance and repair. In both cases, it is important to ensure that there is reliable local access. One solution is the use of local materials – those that are produced from scratch locally – which will contributes directly to the local economy. In many cases, however, it will be overly restrictive to use local materials only, and a second solution is the use of locally-available materials – those that can be acquired locally and have a reliable supply chain to ensure their continued availability. If a material is not available locally, it may also be possible to render it so by establishing a new supply chain.

There is a greater potential for problems in the supply chain for materials used in maintenance and repair since the need for them will be much less frequent and predictable. This should be a consideration in material choice, and especially when establishing new supply chains, which will not survive if there is no regular end consumer. In rare cases where other benefits deem a technology appropriate despite the use of non-locally available materials and no potential for a supply chain, a plan must be established to import materials when needed later in the life of the technology.

Finally, *expertise* here refers to both the knowledge of how to operate the technology and the ability to transfer this knowledge to others, as well as the knowledge needed for maintenance and repair. Typically, the level of expertise needed to operate a technology is less than that needed to repair it, and thus it is expected that the second will more often be the cause of problems. This expertise could be in the form of technical support from development organizations, in which case there is a requirement for these organizations to plan for continued support. If such support is not planned, it is essential that knowledge about how to maintain and repair the technology be transferred along with the technology itself. It must be noted, however, that if the required level of expertise is too high, or if the use of the knowledge is likely to be infrequent, this strategy is unlikely to be effective.

In many cases, the ideal situation would be for the level of expertise required for maintenance and repair to be low enough that these tasks can be accomplished by the regular user of the technology. Thus, the person most familiar with the workings of the technology could be in full control of the technology, which encourages a sense of ownership and increases the likelihood that maintenance and repair tasks will be performed in a timely and responsible manner.   Of course, this is only possible if the complexity and scale of the technology is reduced to a reasonable level.

2.2.3.1.5 - No Serious Adverse Environmental Effects

In an ideal world, human activity would have no impact on the environment. However, in reality much of the human activity in the world has direct negative impacts on the environment. Growing awareness of this impact has driven the developed world to be more environmentally-conscious in all aspects of life, and environmental concerns have risen to a level of primary importance in many spheres. In spite of this, humanity as a whole, and the developed world in particular, has an enormous ecological footprint and has impacted the environment in countless ways, and continues to do so.

Demanding that appropriate technology have no environmental impact establishes an unreasonable standard that even those who advocate it cannot generally reach. This sort of standard can be encouraged, but only if equivalent options are available that do not limit development for the sake of larger environmental concerns.

This is not to suggest that the environment should be ignored, and any serious, irreversible negative environmental impacts must be avoided at all cost. But minor environmental impact has to be considered within the evaluation of a project as a cost that may be a necessary part of the project. This cost should be minimized, as with any other cost, but it is unreasonable to start from an overly constrained position with respect to one aspect of the project.

2.2.3.2 - Desired Elements

2.2.3.2.1 - Positive Environmental Impacts

Beyond simply minimizing environmental impact, it may be possible to have more positive effects on the environment by reducing impacts that are external to the project, or promoting other environmentally sustainable activities. If possible, this should be pursued, provided it does not conflict with or negatively affect any of the primary requirements and does not eliminate other potential desired aspects.

Some examples of potential positive environmental impact include reusing of waste products from other activities, sourcing materials from environmentally-sustainable or environmentally-conscious sources, facilitating energy efficiency and use reduction, promoting or providing renewable energy, or providing education with respect to environmental sustainability or stewardship.

2.2.3.2.2 - Local Empowerment

An appropriate technology is both a product and a body of knowledge that goes into the design, manufacture, and maintenance of the technology. This body of knowledge is directly applicable to the technology, and, if transferred effectively, will be beneficial to the future prospects of the technology by enabling and empowering the people to take ownership. At the same time, the body of knowledge for the one technology will generally contribute to a general skill set that can help enable a variety of activities and facilitate general development.

Delivering a product without the associated body of knowledge can devalue the technology. At the same time, it is generally significantly less effective to try to deliver a body of knowledge without a tangible product to which it is applied. Thus the two components are complementary, with the body of knowledge improving the effectiveness of the technology and the technology providing a concrete example of the practical applications of the body knowledge.

2.2.3.2.3 - Use of Local Materials

It is almost always that case that the materials to be used in appropriate technology should be locally available. This will reduce costs, support the local economy, and ensure that the materials will be available for maintenance and future manufacture. If the materials are not available locally when first planning the project, it may be possible to build a supply chain that will make the materials available when needed. This supply chain can have many indirect positive economic effects, both locally and slightly farther afield, and allow for the possibility of other uses of the material once it is more readily available. These effects make establishing a supply chain a far superior solution to importing materials for sole use in the project.

A subset of locally available materials is local materials, which are a superior choice all other things being equal. Locally available materials that are not local materials simply have a robust supply chain in place where all production and processing is performed elsewhere. Thus, a large part of the cost of these non-local materials is transferred out of the community. Local materials are those that are produced or processed locally, keeping more of the cost in the community and fostering local employment and business.

A focus on purely local materials has been, in the past, a tenet of the appropriate technology, but with time it has become accepted that there are situations where non-local materials are essential to the development of more useful technologies. With the recognition that supply chains can be built and established in an appropriate manner and that they will have their own positive impacts, locally-available materials have become an acceptable common alternative, and a far more common practice.

2.2.3.2.4 - Use of Labour-based Methods

When relevant, consideration should be given to methods that are more labour-intensive, provided this does not conflict with other objectives, particularly economic feasibility. In the developed world, there is often a tendency to choose technologies that specifically reduce labour, since labour tends to be a relatively expensive inputs. In the developing world, however, the cost of labour relative to materials can be significantly lower, which should be accounted for in technology selection. Decreasing the initial capital required for a product by increasing the required labour component is a reasonable trade-off in the developing world that can actually improve the affordability of a technology (Polak 2008).

Often, labour-based methods are ruled out without consideration by those who focus on the efficiency of the technology. However, this is typically only valid if one has a narrow view of the definition of efficiency. Rate of efficiency, a common measure of the efficiency of production, is simply the ratio of value of outputs to value of inputs. A labour-intensive method will almost always lose out to a mechanized process using this assessment, especially if the cost of labour is not adjusted to reflect local conditions. However, this measure of efficiency does not take into account the initial cost of the equipment or the costs of maintenance and repair, both of which will typically be drastically lower for labour-based methods. A more complete measure of efficiency that takes into account these extra investment costs may often find the labour-based methods to be equivalent or even superior to the other options.

Provided the efficiency is close to equivalent, labour-based methods offer other advantages that may overcome any small differences in efficiency. Labour-based methods promote greater employment, which can have significant social and cultural benefits. Labour-based methods often use traditional skills, which can promote, and at the same time take advantage of, an existing cultural knowledge. Finally, the equipment and technology used in labour-based methods tend to be smaller, simpler, and less expensive, which promotes many other beneficial components of appropriate technology, such as sustainability and a sense of ownership.

2.2.4 - Summary

The exact nature and scope of appropriate technology have changed, evolved, and expanded over its history, but the underlying concept has remained relatively constant. This can be summarized by a paraphrase of Conteh's definition which states that the essence of appropriate technology is that the value of a technology must be consolidated by the milieu in which it is to be used. Working from this concept, an assessment framework has been proposed that structures the evaluation of the appropriateness of a technology in a similar manner to any project or technology but with some specific differences and nuances.

There must be a need and desire for the technology, which must be designed or selected to have functional adequacy, economic feasibility, sustainability, and cause no serious adverse environmental effects. In addition, wherever possible, it is desirable to design or select technologies that promote positive environmental effects, local empowerment, use of local materials, and use of labour-based methods.

This general assessment framework can be applied to any appropriate technology, and will be used below to discuss the nature of the technologies to be used in rural transport development.

2.3 - Consideration of Appropriate Technology in Rural Transport Infrastructure

As with any development project, the principles of appropriate technology should be incorporated in the planning for rural transport development. In this section, the focus will be on appropriate technology of rural transportation infrastructure, including the roads, trails, and bridges that make up the transport network.

2.3.1 - Required elements

The required elements of an appropriate technology, as presented earlier, are a need and desire for the technology, functional adequacy, economic feasibility, sustainability, and no serious negative environmental effects.

2.3.1.1 - Need and Desire

Surveys and accessibility studies can be useful to identify problem areas and help formulate priorities, however it is essential that the planning process have community involvement to accurately determine if there is a need and desire for the solution. Furthermore, it is especially important to include women in planning process since they tend to bear the majority of the transport burden and have the most to gain from improvements. In terms of network planning, there is both the question of whether a specific link is wanted or would be used, and what level of service the link can or should provide.

One important aspect of upgrading rural transport infrastructure that is sometimes overlooked is safety. It is sometimes assumed that improving infrastructure will make it safer to use. However, in an area where foot traffic dominates, allowing, or increasing the number of, motor vehicles can make it significantly less safe for those walking. The benefits of increased accessibility will be somewhat balanced by a decrease in safety. This aspect of the debate must be clearly presented to the communities and considered in the planning process.

2.3.1.2 - Functional Adequacy

The major flaw in the highway-and-car approach to rural transport development was that it ignored how the transport network was actually used by the majority of the rural population. Most travel was done off-road and by foot, using extensive trail networks. While roads can still be used by foot traffic and may facilitate travel slightly, the benefit is not reasonable when compared with the level of investment. In many of these cases, the investment could have been more effectively applied in improving the off-road transport network and increasing the availability and use of IMTs. This illustrates a key consideration in appropriate technology, namely that, while it is essential to meet a minimum level of functionality, it can be as important not to overshoot the appropriate level of function.

The first functional goal of rural transport infrastructure should be to provide basic access to as much of the population as possible. This helps enable a minimum standard of living with a particular focus on underserved and isolated communities. The second functional goal will be to reach a target level of accessibility that helps enable and promote socioeconomic improvement.

Determining the appropriate accessibility target is difficult and will need to be based on judgement and experience. Infrastructure should be designed with the appropriate level of function to meet this target while not vastly exceeding it, but must also be assessed within the complete accessibility framework to ensure it will actually effect the desired improvement.

2.3.1.3 - Economic Feasibility

As with any project, there must be money available to pay for any development of the rural transport network. Transport networks are generally relatively large and can be correspondingly expensive. In addition, because of the distributed nature of transport infrastructure, it can be difficult to associate components with specific communities, reducing sense of ownership and contribution of resources. Acquiring adequate resources to fund the project will be a not insignificant task. Because of the magnitude and distributed nature of the system, such resources are likely to have to come from larger organizations, such as governments or aid agencies, as opposed to primarily coming from the local communities.

In addition to initial funding for construction, it is essential that there be funds available for regular maintenance. This was a second flaw of the highway-and-car approach. Even the most expensive and advanced road will need maintenance within a few years of construction, especially in a climate with large seasonal variations in temperature or precipitation. The more expensive the road, the more expensive will be the repairs. In the past, when the roads were built, no funds were assigned for maintenance; it was assumed that the country would be able to shoulder this smaller burden after the initial large investment in construction. However, the maintenance costs of these highways were often significant, consuming resources that could have been used to develop other components of the network instead of maintaining an over-designed component that only benefited a very small portion of the population.

Maintenance costs will be lower for unpaved roads than for paved roads, though they may be significant relative to the cost of the road itself. Lebo and Schelling (2001) write that "maintaining an earth or gravel road is relatively costly. As a rule of thumb, undiscounted maintenance costs over the typical life of RTI will equal the initial construction costs. For example, a typical $5,000/km basic access road may cost an average of $250 a year per km to maintain over its assumed twenty-year life."

2.3.1.4 - Sustainability

Ensuring the continuing function of transport infrastructure can be as much of a challenge as developing the infrastructure in the first place. Unpaved roads and trails, which make up the majority of the rural transport network in developing countries, are susceptible to deterioration, even when properly designed, and require regular maintenance to ensure continued passability. Such deterioration is particularly likely to occur at steep sections, water crossings, and flood-prone areas, and these are the areas where continued maintenance should be focused until such a time that spot improvements can be made.

Lebo and Schelling (2001) report on the "Roads 2000 Program in Kenya" in which a rehabilitation of the unpaved road network was followed with the establishment of a permanent labor-based maintenance system, made up of a number of small contractors each responsible for performing routine maintenance. Such a step was an essential part of the overall plan, and necessary to ensure that the unpaved road system would not deteriorate back to its pre-rehabilitated state.

In addition to maintenance to ensure continued functionality, community involvement in the planning and construction of roads and bridges can help protect the infrastructure from destruction through conflict. Tuladhar (2007) reports on the effect of the Nepali Civil War on footbridges built by Helvetas in the country: "In the 10 years of conflict that reigned in Nepal, about 20 trail bridges were either destroyed or severely damaged. On closer examination it is found that 19 of those were [long span trail bridges] facilitated by the centre and built by civil contractors. Only one community built [short span trail bridge] was destroyed."

The regard for the community-built bridges was a result of the fact that "many insurgents had themselves contributed to bridge building before joining the insurgency. They knew there were no strings attached to the community bridge programme of Helvetas and no funds channelled through authorities and political units." Tuladhar concludes that "all these factors combined imbued community bridges with a high degree of ownership and conflict sensitivity which do not accrue to bridges built by the centre with funds percolating through political units, authorities and contractors."

2.3.1.5 - No Serious Negative Environmental Impact
Due to its scale and distributed nature, rural transportation infrastructure will inherently have environmental impact, and a key consideration in the decision-making process should be to reduce this impact as much as possible. Lebo and Schelling (2001) explain that transport networks will have both direct and indirect impacts. In terms of direct impact, the development of new roads or trails may require the use of previously undeveloped land. The construction and use of the infrastructure will generate dust in the air and increased erosion and sediment runoff, the latter two of which will be of primary concern near all waterways. Finally, improving roads and trails may lead to more use of motor vehicles, which is desirable from an accessibility point of view, but not necessarily from an environmental point of view. In terms of indirect impacts, an extended or improved transport network may open up previously inaccessible, or marginally accessible, areas for development and resource harvesting.

The effects of each of these impacts can be reduced and controlled, though the cost of controlling them to a reasonable level will have to be accounted for in the planning process. All things being equal, the development of existing routes is preferred over new routes that disrupt previously undeveloped land. Controls for erosion and drainage should be included in the design of the system when working near large slopes and waterways. However, it is the indirect impact of facilitating development in new areas that is both the most challenging to notice and predict and yet the most likely to have long-term negative irreversible impact. It is in these situations that a serious cost-benefit comparison must be conducted that accounts for the marginal benefit in accessibility and the potential environmental cost. In such situations, it may actually be desirable to curtail the function of the network to limit large motor vehicles, and instead focus on IMTs that will effectively improve the mobility of rural populations without opening up new resources to large-scale harvesting.

2.3.2 - Desired elements

In addition to the necessary elements for an appropriate technology, there are additional desired elements that should be incorporated if possible due to their broader impact. These include, as described above, positive environmental effects, use of labour-based methods, local empowerment, and the use of local materials.

2.3.2.1 - Positive Environmental Impact
Unfortunately, transportation has significant socioeconomic benefit but cannot generally be considered to have environmental benefit. In fact, transportation can be largely considered to have negative environmental effects that are far outweighed by the social benefits, especially in the context of rural populations. This must be accepted, and is balanced by the hope that, in the long-term, socioeconomic improvement will lead to a situation where environmental concerns can be seriously addressed.

That being said, if any positive effects can be created through transport planning, they should be considered. Examples could include directing traffic and development away from sensitive ecosystems or providing access to more sustainable sources for certain needs or wants. In terms of construction impact, sourcing materials from sustainable renewable sources can help support such efforts, and should be considered as much as possible.

2.3.2.2 - Labour-Based Methods

Rural transport infrastructure is an application that lends itself well to labour-based methods. Such methods have been broadly tested and are generally found to be competitive on an economic basis with equipment-intensive methods, even before taking into account the broader benefits of increased local employment (Tendler 1979; Edmonds et al. 1980; Tajgman and deVeen 1998). Lebo and Schelling (2001) further suggest that the benefits of labour-based methods extend beyond economics to the empowerment of the local community through the acquisition of skills that then have an impact on the sustainability of the infrastructure:

> "The type of work associated with basic access is ideal for labor-based methods. Spot improvement interventions are small-scale and varied, requiring attention to detail, and often do not require heavy construction equipment. In the case of community [rural transport infrastructure], the full involvement of the community gives them the opportunity to acquire the skills for the eventual infrastructure maintenance by labor-based methods. It is important to note that equipment (for example, graders) are seldom available for subsequent maintenance activity for [rural transport infrastructure], a fact that should be planned for at design."

Despite the advantages of labour-based methods, there may be a number of challenges in their application. Many organizations and governments do not have experience with labour-based methods and may be hesitant to deviate from the equipment-intensive methods to which they are accustomed. Furthermore, labour-based methods typically use local labour, which results in many distributed contracts dealing with small sections of the rural road system. This results in a greater burden of organization at the government level when compared with a single contractor equipment-intensive operation. There is also a certain amount of training and equipment that may be necessary to start a labour-based operation where one did not exist before.

2.3.2.3 - Empowerment

As mentioned above, the use of labour-based methods has an impact on the empowerment of the local population. Labour-based methods employ a large number of members of the local population, increasing the number of people who will acquire new skills. The skills transferred through rural transport infrastructure work are based on the use of simple tools and concepts, which can be applied on future transport infrastructure maintenance or transferred to other fields.

2.3.2.4 - Local Materials

As much as possible, rural transport infrastructure should be constructed using local materials. In most cases, this means avoiding steel and concrete except where absolutely needed. The use of local materials helps keep money in the local area, but also has the added effect of facilitating the empowerment of the local population. The skills acquired through working on the development of rural transport infrastructure are likely to be linked to the materials being used. If these materials are local, the skills will be more transferable to other fields in the same location.

2.4 - Appropriate Bridge Technology

Bridge construction in developing countries has two extremes, ranging from simple makeshift bridges built at the community level to satisfy local needs up to large engineered structures built at the governmental level to satisfy national infrastructure needs. The first tend to use simple available materials and are limited in their capacity, span, longevity, and safety due to a lack of engineering. The second are typically well-engineered location-specific solutions that use modern materials. Because of this, they tend to be relatively expensive and as such are typically only an option for a developing nation's primary road network.

Falling between the possibilities of these two extremes are multitudes of crossings and link locations in rural road and trail networks. These crossings are challenging enough to require a minimum level of engineering, and are thus beyond the typical capacity of a rural community, but at the same time are not part of a primary road network. As such, they are not a priority for national governments and, therefore, fall outside the scope of the available funds for transportation infrastructure. As a result, there is a general need for appropriate road and trail bridges that can serve this intermediate function in much of the developing world.

2.4.1 - Bridge-building organizations

A number of organizations or individuals have attempted to satisfy some of this need, though the focus is almost exclusively on trail bridges, since these are often viewed as having the most impact for the most disadvantaged. Some distinguished examples include: Helvetas, Bridges to Prosperity, and Toni Ruttimann.

2.4.1.1 - Helvetas

Helvetas, or the Swiss Association for International Co-operation, is a charitable organization started in Switzerland that now has projects around the world. Helvetas has four working or technical focus areas: infrastructure in rural and semi-urban areas, sustainable management of natural resources, education and culture, and civil society and the state. One aspect of the infrastructure in rural and semi-urban areas focus is the support of local initiatives for the construction of pedestrian trail bridges (Helvetas 2008).

In Nepal, Helvetas has been working with local communities and the Government of Nepal since 1972 to support trail bridge building. The program has seen several major structural changes in its time, initially working only with the government, then changing to a decentralized community-based approach, and eventually settling on a hybrid of the two that maintains decentralization and community involvement while being organizationally supported through the umbrella of the Department of Local Infrastructure Development and Agricultural Roads within the Government of Nepal's Ministry of Local Development. (Tuladhar 2007)

The pedestrian trail bridge program has been incredibly successful, having built more than 3000 bridges in Nepal, and more in Bhutan. Commenting on the effectiveness of the trail bridges, Tuladhar found that:

> "Safety, convenience and time saving are the three invariable, constant and primary impacts of durable bridges. Round the clock accessibility to the other bank at all times allows people to plan their work activities according to their convenience and enables them to respond to emergencies. This psychological advantage is immense but incalculable.
>
> Crossing torrential Himalayan rivers on foot or by means of dugout boats is a risk-filled undertaking. People, animals get swept away and dugout boats overturned when least expected incurring loss of life and dismemberment of limbs. A bridge effectively abolishes such risks. The benefits are incalculable.
>
> A bridge shortcuts long detours to the next nearest crossing where such exists, else diverts traffic towards the trail it connects and saves much time and effort. The bridge at Molung Dobhan in Nepal diverts traffic from the traditional main trail to the district headquarter saving as much as 4 hours for each traveller. Many bridges in the remote corners of Nepal and Bhutan abolish the need to make day long detours over rough trails and terrain.
>
> Accounting only for time savings in pure value terms and in a very conservative manner (at US $ 0.07 per hour), the return on investment of the three bridges surveyed in Nepal is found to be quite impressive – in the range of 18% to a phenomenal 169%. In Bhutan traffic at 23 bridge sites (after construction) were found to increase almost by 100%. The time savings are recorded at 11,748 hours per day equivalent to 528,660 man days per year."

These findings support many of the assertions of the special role bridges can play in the alleviation of transport burden and promotion of socioeconomic improvement.

As part of the decentralization process, bridge guides were developed to provide standardized procedures for the pedestrian bridge models and eliminate the need for design knowledge (Helvetas/TBSSP 2003a; Helvetas/TBSSP 2003b). These short-span trail bridge guidelines cover the construction of cable-supported pedestrian bridges of up to 120 meters in length. There are two standard models, a suspended type where the deck sits directly on cables and a suspension type where the deck is supported below the cable on hangars. Typical profiles for these two types of bridge are shown in Figure 2.4 and Figure 2.5. These guides were a key factor in the success of the program, and can be a useful resource for bridge building in other locations. However, the bridge designs are specifically designed for use in the Himalayas, and as such do contain some design decisions that may not be appropriate for all locations. Thus, some work is generally required to understand the underlying design decisions in order to be able to modify the designs for application in a new location. In addition to cable-supported bridges, Helvetas has also had involvement in the development of standard short-span truss and beam trail bridges (I. T. Transport Ltd. 2004), though to a much lesser extent.

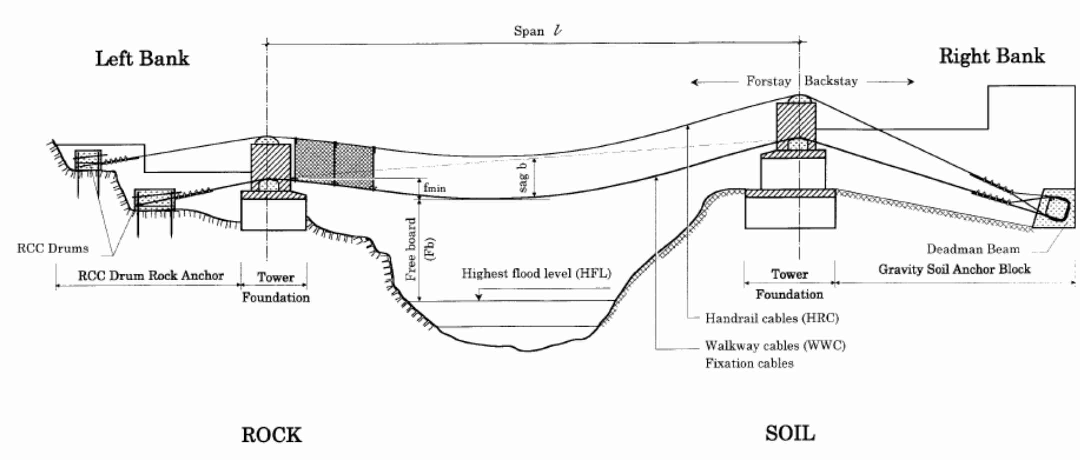

Figure 2.4 - Typical profile for Helvetas pedestrian suspended bridge (Helvetas/TBSSP 2003a)

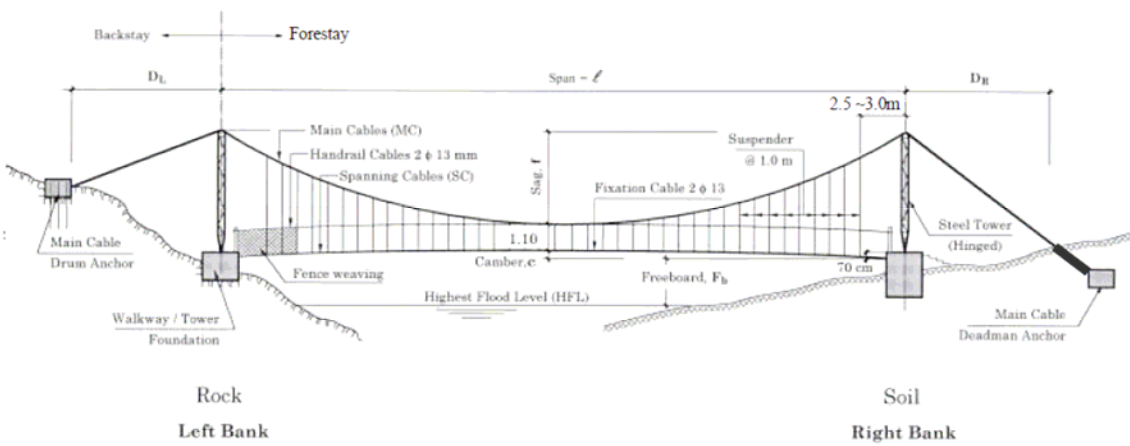

Figure 2.5 - Typical profile for Helvetas pedestrian suspension bridge (Helvetas/TBSSP 2003b)

2.4.1.2 - Bridges To Prosperity

Bridges To Prosperity is a US-based organization that builds footbridges for and with rural communities. Founded in 2001 by Ken Frantz (Bridges To Prosperity 2009), the organization's first bridge was a steel truss bridge built over the Blue Nile River gorge in Ethiopia. Since then, the organization has expanded and taken on bridge building projects in 14 countries located in Africa, Asia, and South America.

Bridges to prosperity works exclusively on footbridges, including short span steel trusses and cable-supported types. Bridges have been built following the Helvetas suspended and suspension trail bridge manuals, with adjustments for regional variations in materials and skills. An example of one of the organization's bridges is shown in Figure 2.6. From their experiences, Bridges to Prosperity have developed their own manual for the planning, construction, and design of the suspended style bridge, their most commonly built type (Bridges To Prosperity 2007).

Figure 2.6 - 65-m Jorogeta Bridge in Ethiopia, built by Bridges to Prosperity in November 2004 (Bridges To Prosperity 2007)

2.4.1.3 - Toni Ruttimann's Rescue Bridges

Toni Ruttimann, also known as Toni el Suizo, is a Swiss-born humanitarian who is personally responsible for constructing more than 300 bridges around the world – 331 as of 2006 (Binney 2006). Toni traveled to Ecuador at the age of 19 after hearing about the effects of an earthquake on the local population. There, with no engineering training, he solicited donations in materials from oil companies and worked with local communities to build his first pedestrian bridge.

After that first bridge, he continued to build bridges in collaboration with Walter Yánez, a local pipe welder, refining and solidifying his suspension style bridge design, an example of which is shown in Figure 2.7. During that time, he also established continuing relationships with a pipe manufacturer and Swiss cable car companies who donate pipes and cables to erect his bridges. To date, he has built bridges in Ecuador, Columbia, Costa Rica, Nicaragua, Honduras, El Salvador, and Mexico, as well as Cambodia and now Vietnam (Ruttimann 2001).

Figure 2.7 - Toni Ruttimann's fourth, and longest, bridge - a 264-m span over the Rio Aguarico, Ecuador (Bruhwiler and Ruttimann 2000)

2.4.2 - Comparison of general delivery methodologies

The majority of the bridges built by these three organizations are cable-supported trail bridges, as mentioned. The cable-supported bridge contains a certain level of technical complexity and, though it may build on some local cultural knowledge of the rope bridge typology, there will need to be a significant level of transfer of knowledge and skill to empower the local population and develop a sustainable bridge building program. The three organizations being discussed approach the knowledge transfer in relatively distinct ways.

The Helvetas approach has been largely based around working with the government of a country to establish a broad, nation-wide bridge building program. The exact nature of the organizational structure has evolved with time, as described above, and is now a largely decentralized approach that includes community involvement at the local level but is supported by an umbrella structure that is part of the national government. In the end, Helvetas has created incredibly successful and sustainable programs in Nepal and Bhutan, but this has required years of effort and expense, and much support and interest from the government. In many ways, this is an ideal long-term result, but it is not clear if the program can be easily replicated in other countries.

The Bridges To Prosperity approach is one of direct community involvement with the dual goals of providing bridges and empowering the local population to build their own bridges. The organization’s mandate is to operate in a country for a maximum of two years during which time they will build bridges and train local engineers and partners. After that time, the organization should scale back to a supporting role and eventually no role at all once it is clear that a sustainable bridge building operation has been established. With this in mind, the organization has a strong education focus that has even included the construction of model bridge sections and components as part of the training process. The Bridges to Prosperity model offers a lot of potential for growth if the establishment of a sustainable program can happen in a two-year period. So far, the organization has handed off programs in Ethiopia and Peru, though in both cases this was to other charitable organizations, specifically Helvetas and Bridge to Prosperity Peru, respectively. Thus, the programs continue, but are not necessarily self-sustaining within the country itself. Despite this, the organization has been successful in establishing bridge building programs from scratch in these countries, which is a remarkable achievement.

Finally Toni Ruttimann focuses solely on the delivery of footbridges to individual communities. Though his activities have expanded and received government support in some cases, there is no effort towards establishing a bridge building program that would function without his involvement. Even in Central America, where his associate Walter Yánez continues to build bridges despite Toni working half-way around the world in Vietnam, the design of the bridges is done by Toni and the details are transmitted to Walter to implement. The general program has been successful in providing bridges for communities, especially considering it is all driven by one individual. However, there is a limit to the growth potential, and no more bridges will be built once Toni is no longer involved.

2.4.3 - General Discussion of Approaches

There are advantages and disadvantages to the delivery methods of each of the organizations discussed above.

The primary, and significant, advantage of the Helvetas method is that it results in the establishment of a robust, sustainable organization with a string institutional knowledge and memory. The major disadvantage is in the time, effort, and resources needed to establish the institutional framework, all of which are initially being diverted away from building bridges. There is also the potential for the scale of the operation to lead to logistical complications and bureaucracy, as well as high-overhead costs.

The Bridges to Prosperity model, if it works as hoped, has the advantage of establishing an effective transfer of knowledge and the creation of organizations or businesses to continue bridge building work. In comparison with Helvetas, Bridges to Prosperity also seems to operate at a level closer to the community. The major disadvantages of the Bridges to Prosperity approach is the potential for a lack of sustainability if the bridge building organizations cannot support themselves organizationally or economically without the presence and support of an umbrella organization.

The major advantage of the Toni Ruttimann approach is that he works directly with the local communities. There is also some advantage to having very little organizational structure, which can lead to efficiency and a lack of bureaucracy. The major disadvantage is that his approach fosters little empowerment of the local community and establishes no independent bridge building entities.

Ultimately, the Bridges To Prosperity approach seems to be the best, at least in theory and for the short-term. The institutionalization at a large-scale within the government, as done by Helvetas in Nepal and Bhutan, is the correct long-term goal, but to strive for this immediately will divert effort and attention away from the technology and the communities being served. At the same time, the Toni Ruttimann approach which focuses solely on the technology as a product being delivered to the local community has a limited institutional framework and lacks a certain level of sustainability. An approach that works to provide bridges while also training people and establishing small businesses and organizations that can continue to build bridges themselves is an appropriate compromise between an institutional and a delivery focus. The short timeframe set by Bridges To Prosperity is solid in concept, though a rigid two-year limit may ignore different circumstances in different countries and should be allowed to adjust accordingly.

2.4.4 - Trail Bridges vs. Road Bridge

As mentioned previously, there are now a number of standardized resources for constructing cable-supported trail bridges including the Helvetas and Bridges To Prosperity guides. Toni Ruttimann has not released any information about the design of his bridges, though from all indications they appear to follow similar design principles to the Helvetas suspension bridge. In addition to cable-supported bridges, there are also published resources that address the construction of foot bridges of shorter span (I. T. Transport Ltd. 2004). These address the basics of beam bridges, using a variety of materials, and simple truss bridges, primarily constructed from small steel sections.

There is work to be done to compile and generalize all of the resources for trail bridge construction and to extend the bridge building activities to other countries. However, there does not seem to be, at this time, a significant need for research into new structural systems for use in footbridges, based on the level of activity that is taking place in the development of footbridges worldwide.

The field of road bridges has significantly fewer resources. Roads, and the bridges that serve them, are frequently viewed as the domain of the government and so development agencies and communities are less likely to address a deficiency. Toni Ruttimann expressed this belief in a 2006 interview:

> "The World Bank and UN build road bridges, but no one builds community bridges. We don't want to do governments' work for them, but we can access the places they can't," he says. "We work with the communities in the corner, that are not important enough for the government to spend money on." (Knutt 2006)

The implicit assumption here is that trail bridges are the only possible community bridge. However, this overlooks the many smaller rural roads that are most useful to the local communities yet are functionally limited by a lack of small-scale road bridges. Helvetas is beginning to encounter this need in Nepal now that many of the communities have adequate connection on the trail network and are looking for further improvement. Tuladhar (2007) writes:

> "After 1990s, rural roads have proliferated into the hinterlands and have become a major sub-sector. Many agencies are involved and there is already a large road network. Four to six months in the rainy season many roads are closed and do not serve their purpose. Communities are now demanding for rural road bridges but there exists no agency that builds such bridges. There is an immediate need to develop cost effective norms, standards and technology for rural road bridges that can be built through the community approach."

This general need for appropriate rural road bridges can be found in many countries of the world, not just those that have robust trail bridge programs. In a mountainous country such as Nepal, trail bridges are of primary concern since road construction and access will be limited. However, trail bridges will not always clearly dominate as the primary need, and there may be an equal need for rural road bridges. It is this general need that is the foundation for this research.

2.5 - Summary and Conclusions

Rural road bridges are an important component of the rural transport network, which is itself a key factor in any strategy to improve rural accessibility. Improved accessibility has been shown to have the potential to contribute significantly to the alleviation of poverty.

In the developing world, it is important to work with appropriate technology that has been designed to fit into its intended environment, and this is true for rural transport infrastructure, including rural road bridges. An assessment framework for appropriate technology has been presented, and will be used to develop and evaluate bridge technology for rural road bridges.

The goal of this background material was to understand what makes an appropriate bridge technology. Ultimately, there are three general factors that must be considered in any appropriate bridge technology. First, a bridge technology must satisfy the required characteristics of any other appropriate technology, demonstrating fulfillment of a need and want, economic feasibility, functional adequacy, sustainability, and causing no serious negative environmental impact. In addition to this, an appropriate bridge technology must be developed within, and contribute to, an overarching plan for general rural transport development and improvement of local accessibility.

Finally, the nature of bridges makes it such that they can have a significant benefit if they incorporate considerations of the desirable characteristics of appropriate technology, and they should do so. In particular, the use of local materials, labour-based methods with local labour, and training and empowering the local people can all compound together to have much broader implications on the local economy.

In the following chapter, the determination of an appropriate bridge technology for the Tshumbe Diocese will be considered. The characteristics of appropriate bridge technology developed herein will be used in assessing and developing an appropriate solution.

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Chapter 3 – An Appropriate Bridge Technology for the Tshumbe Diocese

As discussed in Chapter 2, the key to an appropriate technology is that it takes into account its context. Since conditions and situations vary between and within countries, there is no technology for any application that will always be the most appropriate choice. Therefore, it is not possible to define one building material as the universally appropriate choice. All building materials have advantages, disadvantages, and unique properties that will make them more or less appropriate in a given situation, and an evaluation of appropriateness should be conducted for any new location or project.

In this chapter, the appropriate bridge technology for the Tshumbe Diocese is investigated. First, structural materials for use in bridges are described and compared. It is concluded that all materials have functional advantages and disadvantages, and any could be appropriate in the correct environment. The key factors in determining the appropriate material for use in a location will be availability: availability of the material, availability of tools to work with the material, and availability of labourers that can work with the material. These factors will have a significant impact on the appropriateness of the structural material and will override any functional differences.

The specific situation in the Tshumbe Diocese is presented, and a conclusion is made that a timber bridge system is the appropriate choice. With this in mind, several timber bridge systems are described and assessed, with none offering an ideal solution for increasing the capacity of bridges in the Tshumbe Diocese. The Town lattice truss is thus proposed as a potentially appropriate solution and the structural system will be developed in the following chapters.

3.1 - General Comparison of Building Materials for Use in Bridges

The DFID Footbridges manual (I. T. Transport Ltd. 2004) provides a comparison for different materials used in the construction of footbridges, which can be almost directly applied to the construction of road bridges. A pedestrian loading equivalent to that used to design footbridges must also be applied to road bridges, since part of their function will be to carry foot traffic. As will be shown later, this loading is of the same order of magnitude as that for vehicles, and the resulting structures will have similar main member strength requirements. The major difference between the two types of loading is a change from a large uniformly distributed load for pedestrians to large point loads for vehicles. This has implications on the design of the secondary load carrying members (deck, stringers) and also has implications on stability and stiffness requirements, most significantly ruling out the use of cable-supported structures.

For short spans, up to a maximum of about 12m, there are a variety of documented options for simple road bridges (Transport Research Laboratory 2000). The simplest option is a beam bridge, consisting of prismatic simply-supported components made of either timber, steel, or reinforced concrete.

If the span being crossed is greater than 12m, it will not be possible to use a single beam and a choice must be made between adding enough mid-span supports to create a string of spans that can be crossed with simple beams, or to upgrade to trusses which can have a greater capacity with the same sized members. Timber or steel trusses, or hybrids of the two, made up of smaller sections are generally claimed to have a maximum span of about 25m (80ft). It may also be competitive to use trusses for shorter spans if larger members are not available for beams.

A general discussion of the characteristics of each of the three primary structural materials used in bridges, namely timber, steel, and reinforced concrete, is presented below along with a comparison between the three.

3.1.1 - Timber

The beams in a timber beam bridge can be either solid logs or sawn lumber and are typically overlain directly with a sawn timber deck. Solid logs will tend to have a higher strength than sawn lumber of the same species, but also have a higher possibility of unnoticed defects. Logs should have bark and sapwood removed to improve longevity but this processing can generally be done locally and will require less infrastructure than sawn lumber, which will need a sawmill to create the rectangular sections from whole trees. Logs have the disadvantages of variable cross-sections that make the laying of a level deck difficult, a generally large size that makes them difficult to transport and maneuver, and a roughly round shape that makes inefficient use of material for one-directional bending. Despite having a lower strength and requiring more processing, sawn lumber may be viewed as preferable to logs in bridge applications since they facilitate a level deck and are typically of a more manageable size.

Timber trusses can be constructed from timber sections to increase the possible span beyond that of the sections as simple beams. This will be necessary if adequate cross-sections or lengths are not available. Provided the truss can be erected on-site, transportation of components to the site will be simpler than with larger beams, although the maneuvering of the completed truss over the span will be no simpler. Timber trusses generally need to be constructed from sawn lumber as opposed to logs, and the joints can be complicated to design and fabricate properly. Timber trusses will often require a level of skilled carpentry that is not necessary for beam bridges.

Timber as a material for bridge construction has a number of advantages as an appropriate material. Wood is grown and available locally in many parts of the world and has an almost universal familiarity. As a structural material it has a somewhat low but reasonable strength-to-weight ratio and a relatively high stiffness-to-weight ratio. Wood can be used and shaped in a variety of ways, and much of this can be done by hand using simple tools. Wood also has time-dependent strength properties, being able to support greater loads when applied for short periods of time. This can be taken advantage of in bridges, where the largest live loads will also have limited durations.

Wood also has a number of disadvantages as a structural material. Mechanical properties have large variability both between species and within a given species. Additionally, wood is an orthotropic material with significantly different behaviour depending on the direction of the grain. Furthermore, the properties and behaviour of wood are sensitive to environmental factors, most significantly moisture. The result of these factors is that, though wood can be worked with using simple tools, it can require a certain amount of knowledge and skill to take advantage of the good mechanical properties.

Finally, wood is generally considered to have shorter life than the other structural materials. Unlike steel and concrete, wood is susceptible to decay through biological attack from fungus and insects. This decay can be mitigated through the use of durable species, the application of preservatives, or the control of environmental factors. If possible, the use of durable wood species that have a natural resistance to biological attack is the best option, however these species are typically in high demand and may be difficult to obtain cheaply. Preservatives can be used to protect wood from biological agents. Coverage, penetration, and level of toxicity will all affect the effectiveness of the preservative. It can be difficult to achieve good coverage and penetration of preservatives without industrialized approaches such as pressure treatment, which may necessitate frequent reapplication. It should also be noted that preservative toxicity is not limited to fungus and insects but can affect other biological elements, making the potential for environmental or human impact not insignificant. Finally, environmental factors that enable decay can be mitigated through design. The most significant factor is moisture, a minimum level of which is necessary for decay. If humidity levels are not too high, it may be possible to simply protect structural members from precipitation and moisture in the ground to keep moisture levels below the threshold required for biological attack. It will also be necessary to provide space for airflow to facilitate drying in the case that water were to penetrate to the members.

3.1.2 - Steel

Steel beam bridges will typically take the form of a set of steel I beams, as these tend to be the section with the highest moment capacity, overlain by a timber or concrete deck. As spans increase, sections will become increasingly heavy and difficult to maneuver on-site. They will also become more expensive to obtain and to transport to the rural site. For these reasons steel beams are typically only reasonable as an option for a short range of spans.

As an alternative to beams, steel trusses made up of smaller sections can be used. Typically, these smaller sections will be more readily available than larger beam sections. Additionally, the transportation of these smaller sections to site, either individually or in modules, will be much easier than with a single large section.

Steel has a number of advantages as a structural material. It has high strength-to-weight and stiffness-to-weight ratios and high overall material strength. It is an isotropic homogeneous material and generally has highly predictable properties. Steel also has two common and robust connection methods, bolting and welding, the design and behaviour of which are well understood.

One of the major disadvantages of steel is that the production process is highly industrialized, and is therefore unlikely to be performed locally. Steel is also likely to be imported from other countries, meaning cost may be primarily controlled by external factors of which the local population has no control. However, the non-local production is mitigated somewhat by the fact that steel is a commonly used material in a wide variety of applications, thus there is likely to be a reasonably robust supply chain, at least for commonly used sections. The fabrication of steel components from sections requires a level of mechanization and will need a workshop with power tools to cut, drill, shape, and weld the steel, and these operations require skilled workers.

Steel is generally considered to have a longer life and require less maintenance than wood, though it is susceptible to corrosion, which can have a significant impact over time. Corrosion is the result of oxidation, which is enabled by moisture and can be accelerated due to the presence of chemicals, particularly salts as might be found in coastal locations. Steel is typically protected from corrosion through surface treatments such as painting or galvanization. Painting must be maintained regularly to fix nicks and cracks and is particularly sensitive at joints, where there is generally increased surface area and irregularity. Hot dip galvanizing is an effective long-term corrosion protection, however it requires a molten zinc bath for coating components, which will generally not be an option in rural locations.

3.1.3 - Reinforced Concrete

A reinforced concrete beam will typically be a single monolithic prismatic section that acts both as deck and longitudinal spanning system. Steel reinforcing bars are made into a cage with members running longitudinally and transversely, which is then cast in the concrete to support tension and shear forces. Since wet concrete cannot support its own weight, formwork must be built to support the load until the concrete has cured.

The primary advantage of concrete is its durability. Once it has cured, concrete is hard and generally non-reactive. Thus, concrete structures are generally considered to have longer lives than timber and even steel structures, and will require low maintenance. Concrete is made up of a mix of small, relatively common components, which can be easily transported to site.

Reinforced concrete has a reasonable strength-to-weight ratio and a low stiffness-to-weight ratio, which can lead to the need for a lot of material. Fortunately, most of the components of reinforced concrete are inexpensive - the exception being the cement - keeping it competitive on a cost basis. However, the relatively high dead weight of the structure has implications for the formwork needed to support the material before it can support itself. The formwork will need to either cross the entire span while supporting the dead weight of the entire structure or have mid-span shoring, either of which has the potential to be prohibitive.

While concrete can have good predictable properties, it is highly sensitive to the quality of the proportioning of the mix and the subsequent curing of the concrete. Both of these aspects may be difficult to control and monitor in rural locations without knowledgeable workers. Without proper quality control it is possible for the final properties of the cured concrete to be well below what is expected. To further exacerbate this, if an inadequate mix is used and leads to flaws in the concrete, there is very little that can be done maintenance-wise to upgrade or repair the structure, and the end result may be a significantly shortened functional life for the structure.

Concrete benefits in durability from the monolithic nature mentioned above. However, good continuous mechanical behaviour is dependent on the nature of the placement of the concrete, ideally performed as a single continuous operation. Depending on the size of the structure, however, it may be difficult to maintain adequate production rates without mechanization or the availability of a very large labour force. Construction joints can be created in the structure to allow for subsequent addition after a gap of time, however the design of these joints will be critical and can have a significant impact on the overall behaviour.

3.1.4 - Comparison of Materials

A summary of some of the characteristics of materials and systems for rural footbridges is given in Table 3.1. These results are based on a survey of footbridges and low-volume rural road bridges in developing countries (I. T. Transport Ltd. 2004). As stated earlier, and as will be presented in more detail later in the chapter, loading for footbridges and rural roads bridges are similar enough that the discussion of one is relevant to the other. Specific characteristics that are included in Table 3.1 are based on four selection criteria for footbridges as defined in the source: availability of materials, technical support and special skills needed, life of the footbridge and level of maintenance, and cost of the footbridge. It should be noted that the results presented are based on a broad survey, and while can be considered to be generally true, there may be significant location-specific deviations. This will be particularly true for material availability and cost.

Based on the discussion presented, the only conclusion that can be drawn is that there is no clear winner between timber, steel, and reinforced concrete. The materials are all viable for use in rural road bridges from both a functional and economic perspective. Each has advantages and disadvantages that result in none being clearly the preferred choice in all situations. Decisions will need to be based on project-specific requirements and limitations.

The three primary determining factors in selecting a material will be the availability of the material, the availability of facilities and equipment needed to work with the material, and the availability of labour knowledgeable in the material. Good availability of each will decrease costs and allow more flexibility. Poor availability will increase costs, or render an option unfeasible. It is possible to improve the availability of any of these elements, by developing a supply chain, purchasing equipment, or instituting training programs, respectively. However, there will be a substantial effort and cost associated with any of these improvements, and this needs to be balanced by the corresponding benefits. Since the materials are all competitive in terms of function and cost, if one material is already dominant and has good availability in all three aspects, it is unlikely that it will be worth the cost of introducing one of the other materials.

Table 3.1 - Comparison of structural materials and systems for appropriate footbridges (Reproduced from I. T. Transport Ltd. 2004)

Type of Bridge	Availability of Materials	Technical Support Needed	Life and Maintenance	Cost
Timber Log Beams	May be available from local forests otherwise will be a problem	District technical supervision of local carpenters and community	Logs: 10 to 15 years, decking: 5 to 10 years. Regular maintenance of deck	Initial: Logs low cost if locally available. Sawn timber deck raises cost. Long-term: medium to high
Sawn Timber Beams	May need to be obtained from large timber suppliers	District technical supervision of local carpenters and community	Beams: 10 to 25 years depending on quality of hardwood; decking: 5 to 10 years. Regular maintenance of deck	Initial: Cost of good timber beams will be high. Long-term: medium to high
Sawn Timber Truss	Smaller sections than for beams and more likely to be locally available but quality may not be adequate for construction of a truss	Standard designs need to be developed by qualified engineer. Workshop inputs needed for joint reinforcements.	10 to 25 years or more depending on quality of hardwood; decking: 5 to 10 years. Regular maintenance of truss and deck	Initial: Cost of timber may be lower than for beams but construction cost will be higher. Long-term: medium to high
Steel Beams	Availability likely to be a problem. Scrap beams may be a low cost possibility.	Workshop inputs needed. District technical supervision of local carpenters and community	Steel: 30 to 40 years and longer if well-maintained with repainting every 2 to 3 years; decking: 5 to 10 years. Regular maintenance of beams and deck	Initial: similar to good sawn timber beam bridge. Long-term: medium
Steel Truss	Should be generally available	Workshop construction of standard design. District technical supervision of installation by carpenters and community	Steel: 30 to 40 years and longer if well-maintained with repainting every 2 to 3 years; decking: 5 to 10 years. Regular maintenance of steel, joints, and deck	Initial: similar to sawn timber and steel beam bridges Long-term: medium
Reinforced Concrete	Should be generally available	District technical supervision of construction of standard design. Labour needed with experience of RCC work	At least 50 years. Low maintenance.	Initial: may be higher than timber and steel bridges due to high labour cost Long-term: low

3.2 - The Appropriate Choice of structural material for the Tshumbe Diocese of the Democratic Republic of Congo

When Catholic Relief Services first began working with Bishop Djomo and the Tshumbe Diocese of the Democratic Republic of Congo, bridge building capability was minimal. Makeshift bridges of small logs and branches were erected by the local population to cross waterways, as shown in Figure 3.1. These crossings were unsafe and prone to failure, and useable foot traffic only. Thus, one of the Bishop's primary objectives was the provision of vehicular bridges to facilitate accessibility and the distribution of food.

Figure 3.1 – Photos of a makeshift bridge over the Lotembo River in Tshumbe Diocese of The Democratic Republic of Congo (Catholic Diocese of Tshumbe 2003)

As there was no significant bridge-building capacity, a decision needed to be made about how to proceed. Building construction, the most likely field for relevant knowledge, included walls built of wood-reinforced earth, clay brick, or concrete block and roofs built from simple wooden trusses and covered with thatch, metal, or clay tile. From this it can be seen that there was some knowledge in timber and concrete, and an availability of wood and cement, but no relevant use of steel for construction. Concrete construction focused primarily on the use of unreinforced block construction for walls with some mass concrete used for floor and foundations. Thus there was little experience with the use of reinforcement, and a potential lack of availability of steel rebar. Therefore, despite the fact that the timber knowledge was not directly applicable to bridge construction, timber was selected as the appropriate material for new road bridge construction.

The availability of wood from local sources is one of the primary advantages of using timber for construction. The Democratic Republic of Congo contains the world's second largest rainforest and has a wide variety of tree species from which wood can be harvested for structural applications. Much of the country is covered in tropical rainforest, including most of the Tshumbe Diocese. A map of forest coverage in central Africa, including the Democrat Republic of Congo, is shown in Figure 3.2.

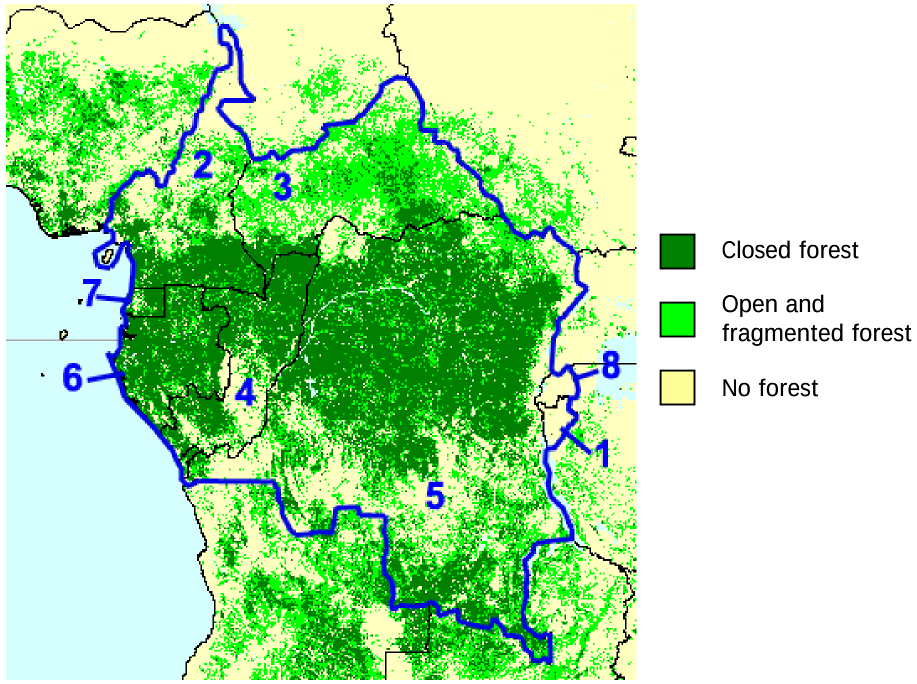


Figure 3.2 - Forest cover map for Central Africa showing Burundi [1], Cameroon [2], Central African Republic [3], Republic of Congo [4], Democratic Republic of Congo [5], Gabon [6], Equatorial Guinea [7], and Rwanda [8] (Food and Agriculture Organization of the United Nations 2001)

Deforestation can be a serious concern in developing countries, typically as a result of expanding agriculture or harvesting of fuel wood. The harvesting of timber for structural use is unlikely to be significant when compared with these two factors, however for our purposes it will still be unwise to promote more timber use if there is a deforestation problem, partially because it will contribute to the problem, but more significantly because the deforestation is likely to reduce the availability of wood and increase cost over a short period of time. In locations where wood is becoming less available for fuel, there is a further danger that wood used in structures might be poached for more pressing subsistence needs. Fortunately, the Democratic Republic of Congo is not currently in the situation of large net loss of forest area, having a change rate between –0.5 and +0.5% per year, as shown in Figure 3.3.

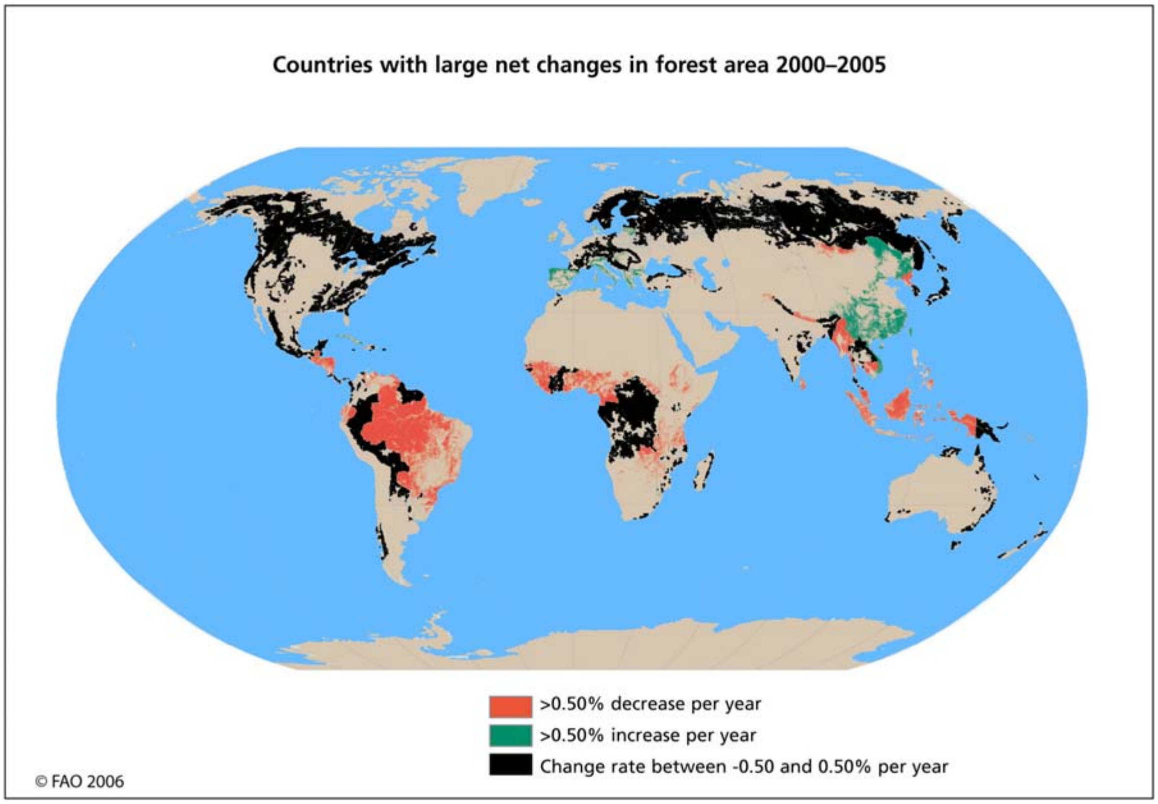


Figure 3.3 - Global map of net change in forest area by country (Food and Agriculture Organization of the United Nations 2006)

The facts that there is current wood availability and likely continued availability for the foreseeable future increase the appropriateness of wood as a building material in the Tshumbe Diocese. The use of local materials is one of the desired characteristics of an appropriate technology as discussed in Chapter 2. The use of a local material has an impact in reducing the cost of materials, improving the sustainability potential, and improving the effectiveness of local empowerment.

To enable the structural use of forest resources, a portable sawmill was provided by Catholic Relief Services. A sawmill makes the production of sawn timber with reliable dimensions from felled trees possible. Such sawmills are powered by electricity, which typically comes from a diesel or gasoline generator, and can cut logs up to 3' in diameter in lengths up to 21', although greater lengths may be possible with auxiliary equipment (Wood-Mizer 2009).

There are a wide variety of wood species available in the Democratic Republic of Congo, many of which could be appropriate for structural use. In July 2005, simple mechanical testing was conducted at MIT on wood samples of five species from the Democratic Republic of Congo: Oleko, Olondo, Olongo, Dihake, and Okolongo. Small wood specimens of each type were provided by Bishop Djomo. Unfortunately, not enough material was available to create enough test specimens to conduct a complete material testing program, but relative stiffness and strength values were drawn from a limited number of tests.

Three specimens of each type of wood, were tested in center-point bending using ASTM D 4761–02a - Standard Test Methods for Mechanical Properties of Lumber and Wood-Base Structural Material (ASTM 2002) as a guide. Specimens were not of adequate size to meet the criteria for any specific testing standard. Specimens were assumed to be comparable with small, clear, straight-grained specimens for comparison with published values (Forest Products Laboratory 1999). Locally available specimens of Douglas fir were also tested using the same procedure for comparison purposes. Final average modulus of elasticity (MOE) and modulus of rupture (MOR) values are given in Table 3.2. Testing results and details on the testing procedure are included in Appendix A.

Table 3.2 - Summary of results for mechanical testing of wood specimens

Species	**MOE** (10^3 psi)	**MOR** (psi)
Okolongo	1724	8833
Oleko	1606	7496
Olongo	1510	7600
Dihake	1146	5528
Olondo	1073	5604
Douglas Fir	1608	5218

Comparing the results for the five wood species, it can be seen that Okolongo is the stiffest and strongest, followed by Oleko and Olongo at a somewhat reduced level, and Dihake and Olondo at a significantly lower stiffness and strength. Comparing with Douglas fir, it can be seen that the first three woods (Okolongo, Oleko, Olongo) have comparable stiffness but significantly higher strength, while the last two (Dihake, Olondo) have significantly lower stiffness but comparable strength. The values for moduli of elasticity found through the testing are thought to be reasonable predictions for the specimens and the values obtained for Douglas fir are comparable with published values. The results for modulus of rupture of Douglas fir are lower than published values and it is unclear if these values are low for the specific type of Douglas fir used in the testing or represents a systematic error in the testing. The modulus of rupture values for all species are low when compared with typical results for small, clear, straight-grained specimens. For this reason, only relative strength between species can be determined.

Okolongo was the wood of choice for the bridges built with the help of Catholic Relief Services. In addition to its high stiffness and strength, Okolongo is also known by the local population to have good durability and resistance to decay. The major trade-off of working with Okolongo is that it is an exceptionally hard wood, giving it good mechanical properties, but making it more difficult to shape and leading to more wear and tear on tools and equipment.

With the use of the sawmill to create the regular sawn timbers, the Diocese gained the ability to build simple timber beam bridges, an example of which is shown in Figure 3.4, replacing the makeshift crossing shown in Figure 3.1. The bridge is made of simple large timber beams running longitudinally, directly overlain with deck planks running transversely. If the span is too great for the lengths and depths of beam available, mid-span supports are provided in the form of logs driven in the middle of the waterway.

Figure 3.4 - Timber beam bridge built in the Tshumbe Diocese of the Democratic Republic of Congo with the assistance of CRS (Catholic Diocese of Tshumbe 2003)

These beam bridges are a vast improvement of the previous options for bridges in the area. However, there are potential limits in span and longevity of the bridges that should be addressed. One of the key methods of controlling decay in wood is the limit the moisture in the wood. Embedding logs in water as mid-span piers is likely to be the primary source of decay in the bridge. This decay will not occur underwater since there is no oxygen for the organisms to breathe, but instead at the surface of the water where there will be constant rewetting of the wood due to water flow and changes in river height. Reducing mid-span supports by increasing the possible span of the structural system could have an effect on the longevity of the system.

Options for timber bridge systems will be discussed in more detail in the following section, from both a functional and an appropriate standpoint.

3.3 - Bridge Systems

There are a variety of different timber bridge systems that have been developed and used for road bridges. Three specific timber bridge systems that were developed for use in developing countries will be considered as precedents and options within this research.

Below, the functional characteristics of rural road bridges, including geometric requirements and loading, will be presented, followed by a discussion and evaluations of each of the bridge types. Finally, the bridge types will be compared and their appropriateness for use in the Tshumbe Diocese will be determined.

3.3.1 - Functional Characteristics

Rural roads in developing countries that are designed for basic access will have extremely low volumes of vehicles, typically well below 50 vehicles-per-day, which is a common threshold value that distinguishes an upper limit on basic access. Such low-volume roads are generally one lane and unpaved, and the bridges that serve them should be designed accordingly. While it will be necessary in the design of the roads to allow for passing of prevailing vehicles, either through adequate width of shoulders or the provision of regular passing places (Lebo and Schelling 2001), it is reasonable for bridges to be sections of the road where passing is not possible. Single-lane bridges are considered adequate for traffic flows up to around 200 vehicles-per-day, since at such flow rates they will not cause serious delay to vehicles (Transport Research Laboratory 2000). To ensure safety, single-lane bridges must have approaches with adequate sight lines and sufficient width for two vehicles.

Rural road bridges should be designed to carry a variety of types of traffic including pedestrian, IMTs, and motorized vehicles. This will affect the required loading, as discussed below, and will also need to be considered in other basic design details. For pedestrian loading and vehicles with small wheels, gaps between deck planks will need to be small. If running boards are provided, they should accommodate axle widths of both IMTs and motorized vehicles.

The nature and quality of a rural road will limit what vehicles can and will use the road and the bridges that service it. A bridge on a smooth wide road near a major highway is likely to see larger loads than a more remote bridge that is serviced by rough narrow roads. Commonly, the largest vehicle that uses rural transport infrastructure will be a seven-ton truck (Lebo and Schelling 2001), and this is consistent with the vehicles known to be used in the Tshumbe Diocese. For reference, an example of a 7.5-ton (7500 kg) truck is shown in Figure 3.5.

Figure 3.5 - Mercedes-Benz Atego 816 7.5 ton truck (mercedes-benz.co.uk)

Finally, a clear width of 3.65 m (12 ft) is recommended for vehicles on single-lane bridges (Transport Research Laboratory 2000).

3.3.2 - Loads

Bridges must be designed to support both vertical loading and horizontal loading. Vertical loading is generally related to the primary function of the bridge, i.e. supporting load over a span, while the horizontal load is generally incidental loading such as wind and impact.

The different types of loading will need to be combined together to determine the total load on the bridge. Load combinations for timber bridges will be discussed in Section 3.3.2.3.

3.3.2.1 - Vertical Loads

Vertical loading will include dead load consisting of the self-weight of the bridge, as well the functional loading of the bridge, which includes pedestrian loading and vehicle loading. Vertical loading can also other live loads, such as snow, however this is not considered relevant for this particular analysis.

3.3.2.1.1 - Dead Load

The dead load of the structure is based entirely on the unit weight and amount of material used in the bridge. Specific gravities of a variety of wood species are known and can be used to find a unit weight, although it will need to be adjusted to account for moisture content. Alternatively, AASHTO suggests a standard unit weight for timber of 50 pcf (800 kg/m^3) for highway bridges (AASHTO 2002), which is considered to be a conservatively high but prudent value to use for most timber bridges (Pierce et al. 2005). This value assumes an average weight structural wood with a high moisture content as would be seen in a wooden support structure that is exposed to the elements.

The amount of material in a given structural system can be derived from geometry, and this is combined with the unit weight to yield dead load from self-weight. Resulting loads should be treated as uniform pressures or resolved into equivalent line loads as appropriate for the members being analyzed.

3.3.2.1.2 - Functional Load

The functional load on the bridge is made up of two non-concurrent loading conditions: pedestrian loading and vehicular loading. Because of the planned limited width of the bridge, a combination of the two loads is unlikely or impossible. In the analysis and design of typical highway bridges in the developed world, pedestrian loading is not considered as an alternate to vehicle loading since most highway bridges will never see gatherings of pedestrians. Full pedestrian loading is typically only considered likely for bridges that are known to be closed to traffic for pedestrian use, such as the Verrazano Narrows Bridge which carries the New York City Marathon, or the Longfellow Bridge in Boston which is used as a vantage point for Fourth of July fireworks.

In rural areas of developing countries, a large amount of travel is carried out on foot, as discussed in Chapter 2. Rural road networks and the bridges that serve them may see both maximum vehicular loading and maximum pedestrian loading and must be designed to support both, though not concurrently.

3.3.2.1.2.1 - Pedestrian Loading

Pedestrian loading will be taken as 0.085 ksf (4 kPa), as recommended by AASHTO for bridges that will be loaded primarily by pedestrian and/or bicycle traffic (AASHTO 2004). This results in loads slightly larger than the pedestrian loading of 400 kg/m^2 recommended in the construction of community footbridges (I. T. Transport Ltd. 2004). This loading, when applied as a uniform pressure over a deck clear width of 12' (3.65 m), yields an equivalent line load of 1.02 kip/ft (14.6 kN/m).

3.3.2.1.2.2 - Vehicular Loading

Rural road bridges should be designed to support the largest vehicle that they will see over their lifetime. It is important that they not be underdesigned, which might lead to a failure, but it is also important that they not be significantly overdesigned, which will result in inefficient use of limited economic resources. It is critical to determine an appropriate design vehicle that will yield a safe bridge while representing a realistic level of required functionality.

Recent versions of AASHTO highway bridge standards (AASHTO 2004) require the use of a single design vehicle, in the past designated as HS20-44, where H means a highway loading, S means a semi-trailer type vehicle, 20 means a gross vehicle weight (GVW) of 20 short tons (40000 lbs) for the first two axles, and 44 means 1944, the year the loading was first designated. The standard HS20-44 truck is shown in Figure 3.6. Axle loads should be divided to the wheels as appropriate and the length of the trailer is varied between 14 feet and 30 feet to determine the maximum possible loading condition.

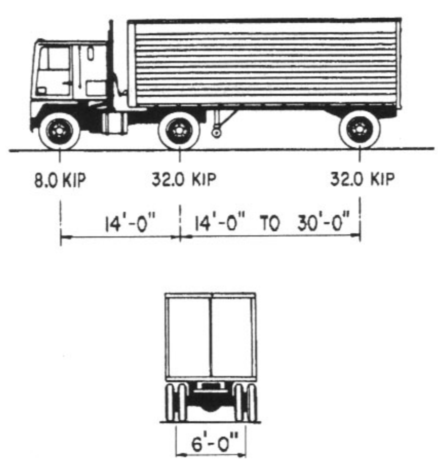


Figure 3.6 - Characteristics of the design truck from *AASHTO LRFD Bridge Design Specifications* (AASHTO 2004)

Rural roads are unlikely to ever see a semi-trailer type truck due to limitations on width, clearance, and bearing capacity. Past versions of AASHTO standards included H loadings that represent a 2-axle truck, which is more realistic for rural roads. Three value for GVW were originally included in the standard, H20-44, H15-44, and H10-44, though the 10-ton design truck was not included in later revisions. The standard truck for H loadings is shown in Figure 3.7. The load is distributed 20% to the front axle and 80% to the rear axle, and this distribution will also be true for the H10-44 bridge, yielding 4000 lbs on the front axle and 16000 lbs on the rear axle.

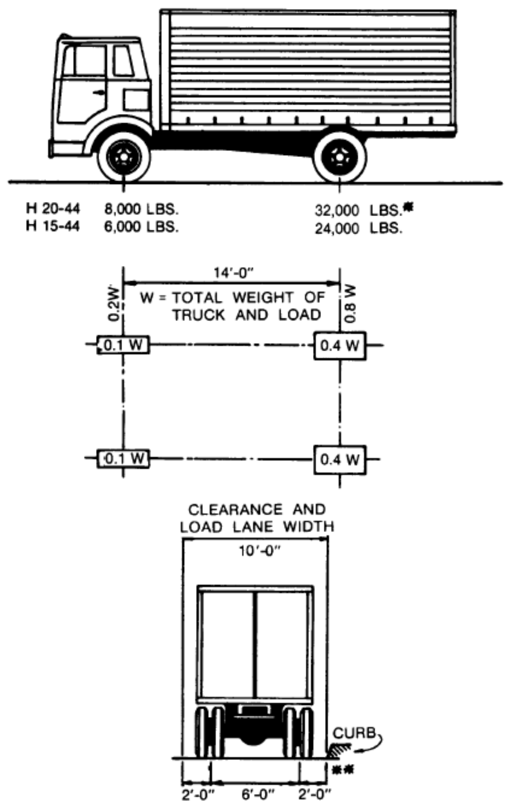


Figure 3.7 - Standard H trucks from *AASHTO Standard Specifications for Highway Bridges* (AASHTO 2002)

In addition to individual design trucks, which will be run across a single lane bridge individually, past versions of AASHTO also required the use of a design lane load, which would represent a lane full of vehicles. The loading is made up of a distributed line load with a single point load that is applied at the location that yields maximum effect. This is intended to simulate a string of lightly-loaded design trucks as well as a single heavy axle. The lane load values for H20-44 and HS20-44 loadings are shown in Figure 3.8. Values for H15-44 and H10-44 loadings will be 75% and 50%, respectively, of the design values shown.

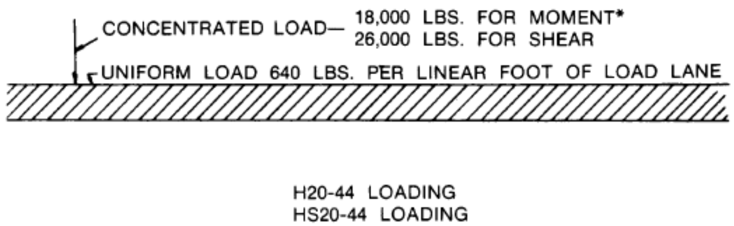


Figure 3.8 - Lane loading from *AASHTO Standard Specifications for Highway Bridges* (AASHTO 2002)

For a given bridge, the choice of design truck or lane load will be determined by which yields the maximum stress. For a single-lane simply-supported span, these results can be evaluated without regard to the specific geometry of the bridge, and AASHTO provides tables to determine the dominant type of load and value for maximum moment and shear for spans from 1 to 300 feet. These tables will be referenced herein to determine appropriate live load bending moment and shear force values for bridges that are designed to carry HS20-44 loading. A similar table can be derived for H10-44 loading and is presented as Table 3.3. The values contained will be used herein to determine live load bending moment and shear force values for concurrent H10-44 and pedestrian loading.

3.3.2.1.2.3 - Comparison of Functional Loading

Given that the maximum reasonable vehicle size for remote rural areas such as the Tshumbe Diocese is a seven-ton truck, the H10-44 design loading is considered a reasonable conservative option. Design moments and shear forces are single-lane simply-supported spans carrying H10-44 loading and pedestrian loading are shown in Figure 3.9 and Figure 3.10, respectively.

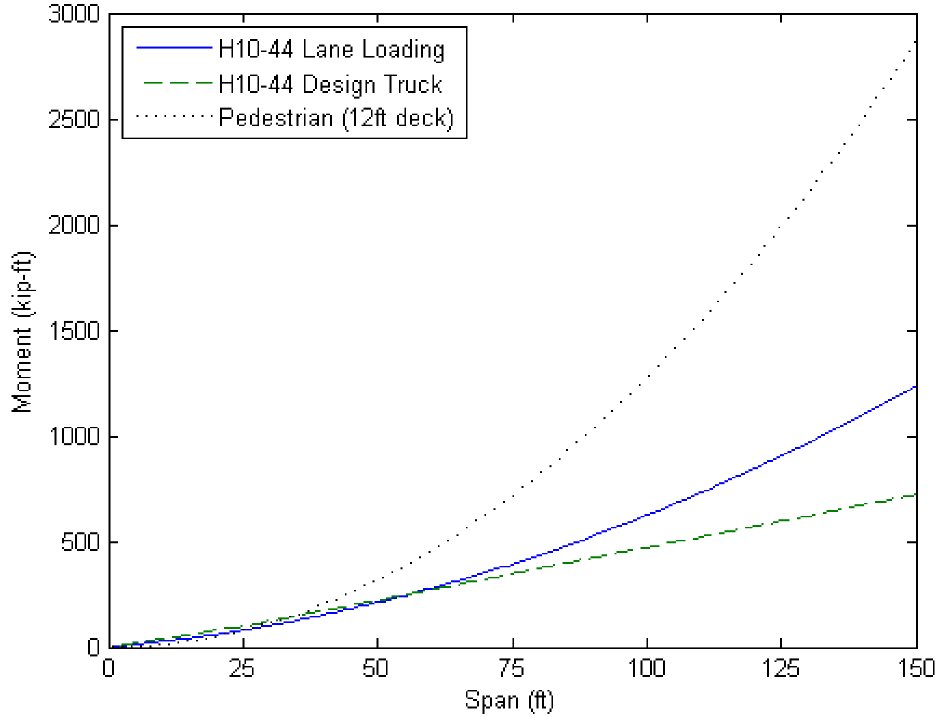


Figure 3.9 - Design moment vs. span for single-lane simply-supported bridges with AASHTO H10-44 vehicular loading and AASHTO pedestrian loading

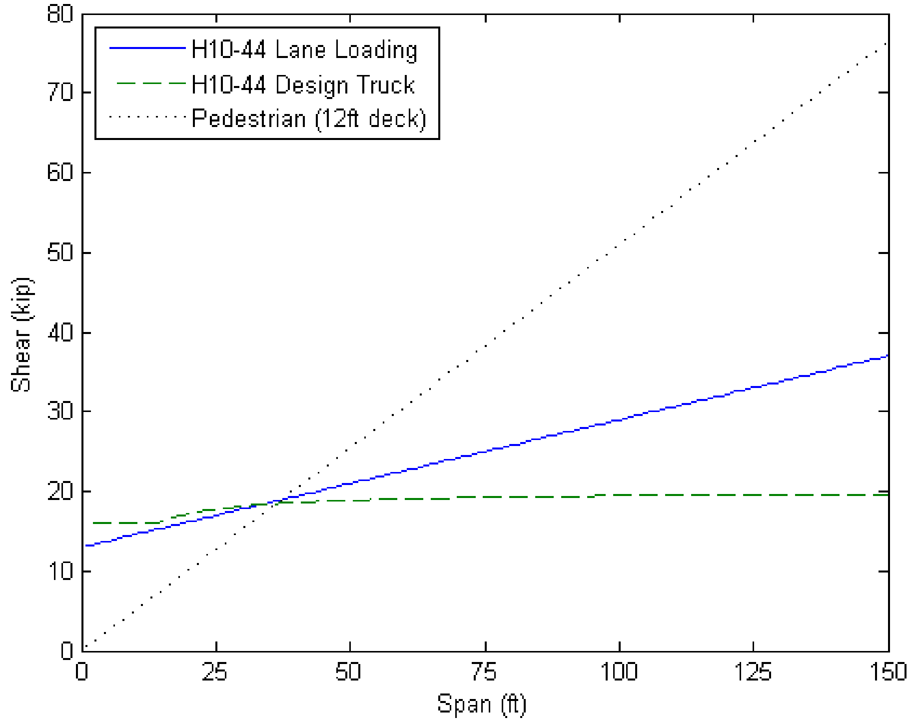


Figure 3.10 - Design end shear and reaction vs. span for single-lane simply-supported bridges with AASHTO H10-44 vehicular loading and AASHTO pedestrian loading

Pedestrian loading controls the design moment for spans greater than 33 ft and the design shear for span greater than 38 ft. Table 3.3 gives the design moment and end shear, as well as the dominant loading type, for spans up to 100 ft that are designed to satisfy non-concurrent pedestrian and H10-44 loading.

3.3.2.2 - Horizontal Loads

Wind loading and flood impact loading will be important considerations in design detailing, but are not considered to be the primary consideration in selecting a structural system. Therefore, they will need to be checked and the system will need to be adjusted accordingly, but they will not be the primary focus of this research.

Table 3.3 - Design moment and end shear for single-lane simply-supported beam bridge based on AASHTO H10-44 loading. Letters indicate dominant load type: (a) Pedestrian loading; (b) Lane load; (c) Design truck

Span (ft)	Moment (kip-ft)	End Shear (kip)	Span (ft)	Moment (kip-ft)	End Shear (kip)	Span (ft)	Moment (kip-ft)	End Shear (kip)
1	4.0 (c)	16.0 (c)	34	147.4 (a)	18.4 (b)	67	572.4 (a)	34.2 (a)
2	8.0 (c)	16.0 (c)	35	156.2 (a)	18.6 (b)	68	589.6 (a)	34.7 (a)
3	12.0 (c)	16.0 (c)	36	165.2 (a)	18.8 (b)	69	607.0 (a)	35.2 (a)
4	16.0 (c)	16.0 (c)	37	174.6 (a)	18.9 (b)	70	624.8 (a)	35.7 (a)
5	20.0 (c)	16.0 (c)	38	184.1 (a)	19.4 (a)	71	642.7 (a)	36.2 (a)
6	24.0 (c)	16.0 (c)	39	193.9 (a)	19.9 (a)	72	661.0 (a)	36.7 (a)
7	28.0 (c)	16.0 (c)	40	204.0 (a)	20.4 (a)	73	679.5 (a)	37.2 (a)
8	32.0 (c)	16.0 (c)	41	214.3 (a)	20.9 (a)	74	698.2 (a)	37.7 (a)
9	36.0 (c)	16.0 (c)	42	224.9 (a)	21.4 (a)	75	717.2 (a)	38.3 (a)
10	40.0 (c)	16.0 (c)	43	235.8 (a)	21.9 (a)	76	736.4 (a)	38.8 (a)
11	44.0 (c)	16.0 (c)	44	246.8 (a)	22.4 (a)	77	756.0 (a)	39.3 (a)
12	48.0 (c)	16.0 (c)	45	258.2 (a)	23.0 (a)	78	775.7 (a)	39.8 (a)
13	52.0 (c)	16.0 (c)	46	269.8 (a)	23.5 (a)	79	795.7 (a)	40.3 (a)
14	56.0 (c)	16.0 (c)	47	281.7 (a)	24.0 (a)	80	816.0 (a)	40.8 (a)
15	60.0 (c)	16.3 (c)	48	293.8 (a)	24.5 (a)	81	836.5 (a)	41.3 (a)
16	64.0 (c)	16.5 (c)	49	306.1 (a)	25.0 (a)	82	857.3 (a)	41.8 (a)
17	68.0 (c)	16.7 (c)	50	318.8 (a)	25.5 (a)	83	878.4 (a)	42.3 (a)
18	72.0 (c)	16.9 (c)	51	331.6 (a)	26.0 (a)	84	899.6 (a)	42.8 (a)
19	76.0 (c)	17.1 (c)	52	344.8 (a)	26.5 (a)	85	921.2 (a)	43.4 (a)
20	80.0 (c)	17.2 (c)	53	358.2 (a)	27.0 (a)	86	943.0 (a)	43.9 (a)
21	84.0 (c)	17.3 (c)	54	371.8 (a)	27.5 (a)	87	965.1 (a)	44.4 (a)
22	88.0 (c)	17.5 (c)	55	385.7 (a)	28.1 (a)	88	987.4 (a)	44.9 (a)
23	92.0 (c)	17.6 (c)	56	399.8 (a)	28.6 (a)	89	1009.9 (a)	45.4 (a)
24	96.0 (c)	17.7 (c)	57	414.3 (a)	29.1 (a)	90	1032.8 (a)	45.9 (a)
25	100.0 (c)	17.8 (c)	58	428.9 (a)	29.6 (a)	91	1055.8 (a)	46.4 (a)
26	104.0 (c)	17.8 (c)	59	443.8 (a)	30.1 (a)	92	1079.2 (a)	46.9 (a)
27	108.5 (c)	17.9 (c)	60	459.0 (a)	30.6 (a)	93	1102.7 (a)	47.4 (a)
28	113.4 (c)	18.0 (c)	61	474.4 (a)	31.1 (a)	94	1126.6 (a)	47.9 (a)
29	118.4 (c)	18.1 (c)	62	490.1 (a)	31.6 (a)	95	1150.7 (a)	48.5 (a)
30	123.3 (c)	18.1 (c)	63	506.1 (a)	32.1 (a)	96	1175.0 (a)	49.0 (a)
31	128.3 (c)	18.2 (c)	64	522.2 (a)	32.6 (a)	97	1199.6 (a)	49.5 (a)
32	133.2 (c)	18.3 (c)	65	538.7 (a)	33.2 (a)	98	1224.5 (a)	50.0 (a)
33	138.9 (a)	18.3 (c)	66	555.4 (a)	33.7 (a)	99	1249.6 (a)	50.5 (a)
						100	1275.0 (a)	51.0 (a)

3.3.2.3 - Load Combinations

Two different load combinations will be considered for the assessment of structural bridge systems: dead load (DL) alone and dead load plus live load (DL + LL). The reason these need to be treated separately, and DL + LL cannot be considered to always be greater than DL, is because of the capacity of wood to resist different levels of load for different load durations. Default design values for wood are based on a ten-year load duration, associated with occupancy live loads. Load duration factors (C_D) for wood from the National Design Specification for Wood Construction (NDS) are given in Table 3.4 and similar values from AASHTO are given in Table 3.5.

Table 3.4 - Frequently used wood load duration factors (AF&PA 2005)

Load Duration	C_D	Typical Design Loads
Permanent	0.9	Dead Load
Ten years	1.0	Occupancy Live Load
Two months	1.15	Snow Load
Seven days	1.25	Construction Load
Ten minutes	1.6	Wind/Earthquake Load
Impact	2.0	Impact Load

Table 3.5 - Timber load duration factors (AASHTO 2002)

Load Duration	C_D
Permanent	0.9
2 months (vehicle live load)	1.15
7 days	1.25
1 day	1.33
5 minutes	1.65

AASHTO recommends a load duration factor of 1.15 for vehicle live loading. This corresponds with a load duration of two months, which is equivalent to that suggested for snow loads by NDS. This seems to be a long duration to associate with vehicle loading, however it must be remembered that AASHTO guidelines are intended for highway bridges, some of which may see relatively high traffic volumes over long, mostly continuous, periods of time. For rural bridges in developing countries, it is expected that the maximum loading, corresponding to a full pedestrian loading or a fully loaded truck, will occur only infrequently and for a very short period of time. Thus, a two month load-duration is likely to be over-conservative. A one day time duration seems realistic, but, to account for a certain level of uncertainty, a seven day load duration value is recommended, corresponding to a load duration factor of 1.25. This seems to be a reasonable compromise between AASHTO recommended values and expected loading.

The dominant load combination can be determined by comparing the resulting values when each combination has been divided by the load duration factor of the shortest component of the loads (Pierce et al. 2005). Thus, the greater of DL/0.9 and (DL + LL)/1.25 will dominate in the primary analysis of timber bridges. In most cases the combination of dead and live load will dominate, and this will be the case as long as the live load is greater than or equal to 40% of the dead load.

3.3.3 - Material Properties

In developing timber beam bridge designs for developing countries, the Transport Research Laboratory (2000) present permissible stress values for use with different wood types. These values are presented in Table 3.6. Heavy hardwoods are defined as having a specific gravity (SG) greater than 0.65 at 18% moisture content (MC), lighter hardwoods are defined as hardwoods having an SG less than 0.65 at 18% MC, and softwoods are a subset of softwoods with an SG greater than 0.42 at 18% MC, which are those considered suitable for bridge construction.

Table 3.6 - Permissible short-term stresses for wood groups (Transport Research Laboratory 2000)

Design Values: MPa (ksi)	Group A: Heavy Hardwoods	Group B: Lighter Hardwoods	Group C: Softwoods
Bending	15.1 (2.19)	8.6 (1.25)	5.4 (0.78)
Tension	9.0 (1.31)	5.0 (0.73)	3.2 (0.46)
Compression parallel to the grain	11.3 (1.64)	6.8 (0.98)	5.0 (0.73)
Compression perpendicular to the grain	2.2 (0.32)	1.8 (0.26)	1.5 (0.22)
Shear parallel to the grain	2.2 (0.32)	1.1 (0.16)	0.9 (0.13)

Douglas fir is considered to be a member of the softwoods group, and the design values given for group C are generally close to tabulated values for No.2 Douglas fir as given in the NDS supplement: design values for wood construction (AF&PA 2005). Based on this and the experimental results presented above, it is reasonable to treat the strongest woods from the Tshumbe Diocese as lighter hardwoods (Group B). It is thought that Okolongo, at least, might be more appropriately classified as a heavy hardwood, but without further evidence it is conservative to assume a lower species group. Therefore, the design values for lighter hardwoods will be taken as reasonable design values for timber bridges in the Tshumbe diocese.

Working stresses were also presented by Parry (1981) for the Kenyan Low-Cost Modular Timber Bridge, which is one of the systems to be presented below. These values are given in Table 3.7. It can be noted that the values for timber correspond roughly with the values defined for Group C in Table 3.6. These values are intended for structural softwood over a density of 650 kg/m3 at 18% moisture content, corresponding to the unit weight used for calculating dead load above.

Table 3.7 - Working stresses for Kenyan Bridge (Parry 1981)

Design Values: MPa (ksi)	Timber
Bending, F_b	5.2 (0.754)
Tension, F_t	3.6 (0.522)
Compression parallel to the grain, F_c	5.0 (0.725)
Compression perpendicular to the grain, $F_{c\perp}$	1.16 (0.168)
Shear parallel to the grain, F_v	0.66 (0.096)
	Steel
Tension, σ_y	147 (21.3)

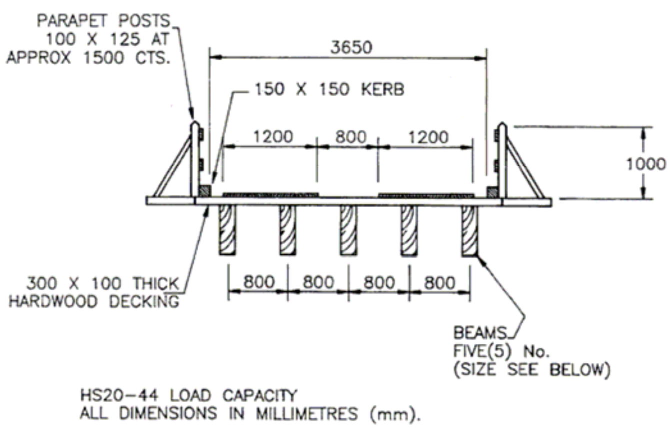
3.3.4 - Timber bridge systems

Three timber bridge systems are presented below, the timber beam bridge, the Allotey built-up girder, and the Kenyan Low-Cost Modular Timber Bridge.

3.3.4.1 - Timber Beam Bridge

The primary feature of a beam bridge is that load is carried as bending moment in the primary members. Beyond this, there are many different arrangements, geometries, and forms that can be used to carry a specific load over a specific span.

For developing countries, rectangular sawn timber sections will typically be used since they are one of the simplest forms. An example single-lane cross-section is shown in Figure 3.11, with dimensions given for a variety of spans and wood types, corresponding to the groups presented in Table 3.6. The bridge shown is designed to support AASHTO HS20-44 loading and uses five support beams to do so. The number of beams used is somewhat arbitrary, and a design could be developed for a different number of beams, though this might involve a deck redesign in addition to a change in beam dimensions.



SPAN (m)	BEAM SIZE FOR TIMBER GROUP		
	GROUP A	GROUP B	GROUP C
4	150 X 375	150 X 500	200 X 550
6	150 X 475	200 X 550	200 X 700
8	200 X 500	200 X 650	250 X 750
10	200 X 600	250 X 725	300 X 850
12	200 X 700	250 X 850	300 X 1000

Figure 3.11 - Standard single-lane sawn timber beam bridge (Transport Research Laboratory 2000)

3.3.4.1.1 - Analysis

The two primary components that must be checked in analyzing or designing a bending beam are the maximum bending stress and the maximum shear stress. For a rectangular beam, these stresses can be calculated using standard formulae.

$$f_b = \frac{6 \cdot M}{b \cdot d^2}$$

and

$$f_v = \frac{3}{2}\frac{V}{b \cdot d}$$

where f_b is maximum bending stress, f_v is maximum shear stress, b is beam width, d is beam depth, and M and V are the internal bending moment and shear force.

The appropriate bending moment and shear force to use for a given beam will depend on how much of the applied loading is supported by a given beam. For distributed loads, such as dead, pedestrian, and uniform lane loading, the load carried by each beam can be determined using a tributary area approach. For design vehicle point loads, also called wheel-line loads, the amount that must be taken by each beam will depend on the properties of the deck. A stiff deck will distribute a point load to multiple beams while a soft deck will allow a point load to be supported by a single beam.

3.3.4.1.2 - Results

The design presented in Figure 3.11 was analyzed and compared with permissible values. Results for Group B wood are shown in Table 3.8. Ratios of design stress to permissible stress for the cross-sections are given, and it can be seen that the values are well below the required value of 1.0. Since the bending ratio is relatively constant and greater than the shear ratio, this is taken to be the governing design parameter. Further details regarding the calculations are provided in Appendix B.

Table 3.8 - Results for analysis of timber beam bridge for HS20-44 loading using Group B wood

	Group B - HS20-44			
Span (m)	b (mm)	h (mm)	f_b/F_b	f_v/F_v
4	150	500	0.57	0.56
6	200	550	0.57	0.51
8	200	650	0.60	0.52
10	250	725	0.60	0.42
12	250	850	0.60	0.39

The cross-sections given in Figure 3.11 are based on an AASHTO HS20-44 loading. Since, an H10-44 loading is considered reasonable for the Tshumbe Diocese, these cross-sections will be significantly over-designed, and it is of interest to develop similar designs for an H10-44 loading. Results for such a redesign are given in Table 3.9. Bending ratio was kept to a similar level to that used for the HS20-44 loading. Further details regarding the calculations are provided in Appendix B.

Table 3.9 - Results of design of timber beam bridge for H10-44 loading using Group B wood

	Group B - H10-44			
Span (m)	b (mm)	h (mm)	f_b/F_b	f_v/F_v
4	150	350	0.62	0.42
6	150	400	0.61	0.42
8	200	425	0.60	0.33
10	250	475	0.57	0.27
12	250	575	0.58	0.26

3.3.4.1.3 - Conclusions

Advantages:
- Simplicity: beams are single saw-cut members

Disadvantages:
- Natural limits to span based on available material lengths and cross-sections
- Large members needed for longer spans may be difficult to transport to site

3.3.4.2 - Allotey Built-up Timber Girder

The goal of a built-up section is to increase the capacity, and therefore the span, beyond that of the largest available section. Moment capacity is increased by increasing the depth of a section and/or moving material away from the centre of the section. A built-up secton is assumed to still behave largely like a beam in terms of the determination of maximum stresses.

The Allotey Built-up Girder is a system proposed and developed by Isaac A. Allotey in conjunction with the Virginia Polytechnic Institute and State University, Blacksburg VA, and the Building and Road Research Institute, Kumasi, Ghana (Allotey 1988; Allotey 1990; Allotey and Dolan 1996). The girder is intended for use in timber-rich developing countries to support bridges with spans exceeding those possible with the available sections.

The girder consists of longitudinal flange elements separated from the center of the section and connected by a solid vertical web consisting of two layers of diagonal members, one layer mirroring the other. The flange elements are intended to carry the bulk of the bending moment and the web diagonals are intended to carry the bulk of the shear force. All members are connected together mechanically using single bolts a the centre of all intersections. Schematic views of the truss are shown in Figure 3.12 and a three-dimensional rendering of the truss is shown in Figure 3.13.

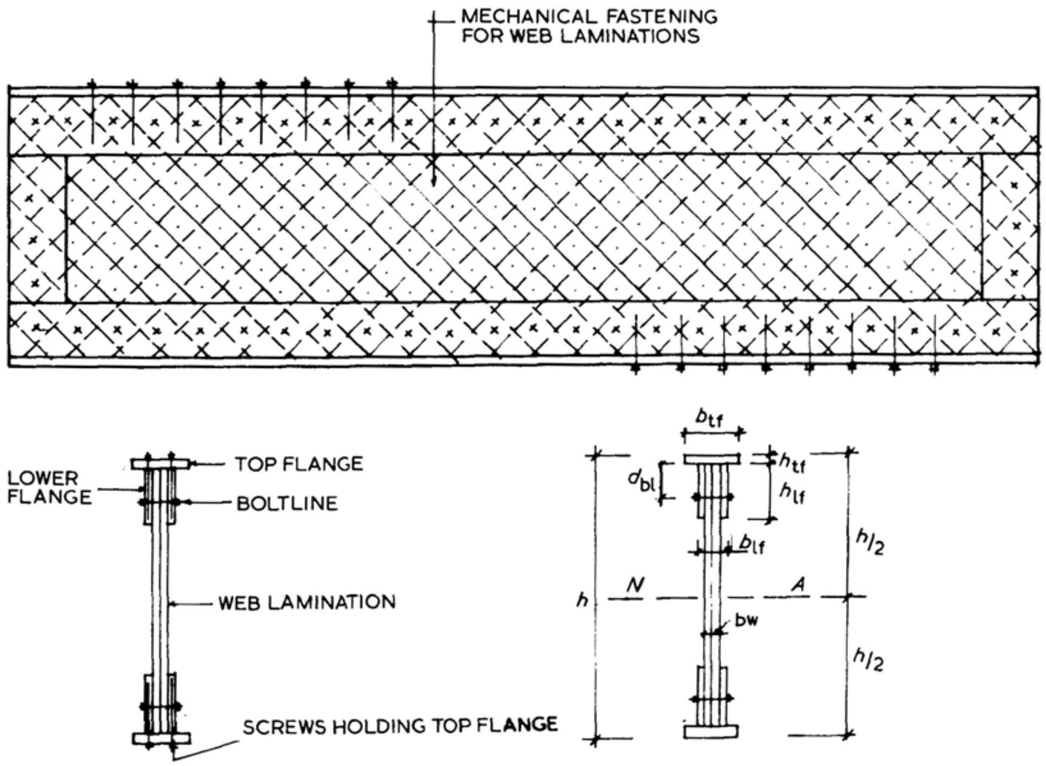


Figure 3.12 - Schematic views and notation for Allotey Built-up Timber Girder (Allotey 1988)

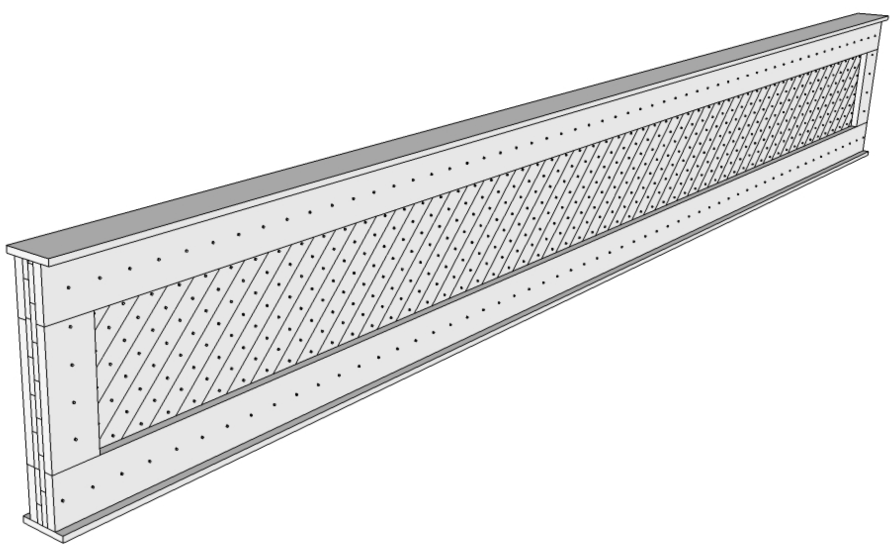


Figure 3.13 - Model of Allotey Built-up timber Girder

3.3.4.2.1 - Analysis

Allotey proposes the use of standard beam bending and shear stress equations, with slight modifications to account for the non-continuous nature of the web. The maximum tension stress on the flange due to bending can be calculated as:

$$f_{t,f} = \frac{M \cdot h}{2 \cdot I_e}$$

where h is the total height of the girder and I_e is an equivalent moment of inertia for the entire sections, calculated as:

$$I_e = I_w \cos^4 \mu + I_f$$

where I_w and I_f are the moments of inertia of the web and flange areas around the centre of the overall sections and μ is the angle of the diagonal web members from horizontal. It should be noted that this formulation assumes the same timber is used for both web and flange elements.

The basic geometry that underlies the determination of the effective moment of inertia is shown in Figure 3.14.

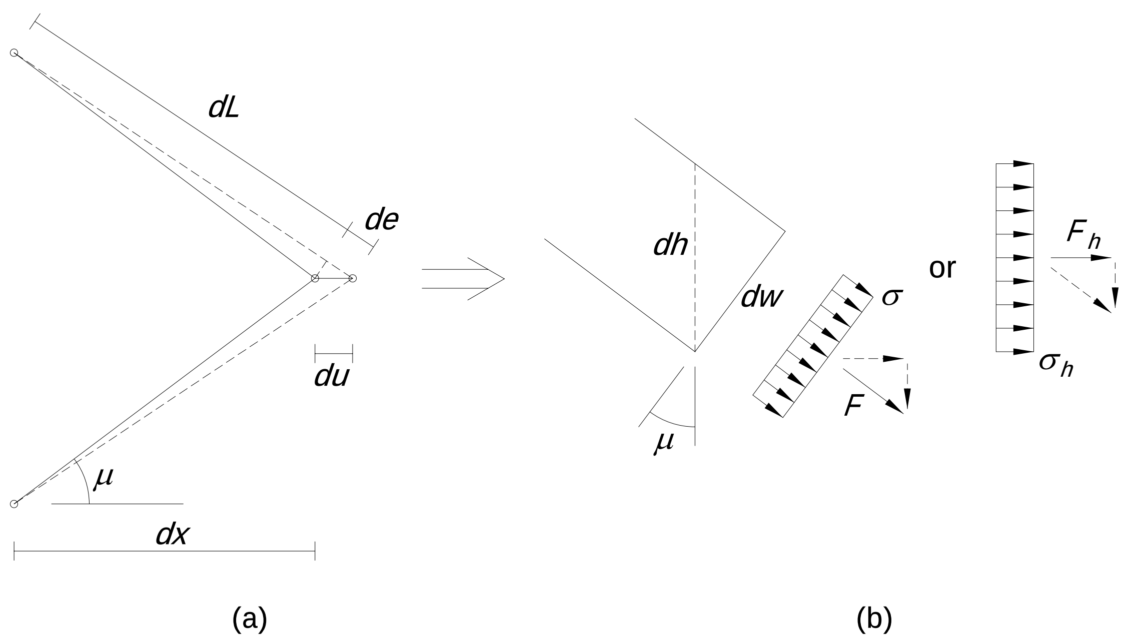


Figure 3.14 - Schematic of differential elements within web of Allotey built-up girder showing (a) deformation of angled members when subjected to a horizontal displacement and (b) stresses within angled members on a cut transverse to the member and a cut transverse to the section

The axial strain that results in the angled members from the horizontal deflection at a given point in the cross-section can be calculated as:

$$\varepsilon = \frac{de}{dL} = \frac{du \cdot \cos \mu}{dx / \cos \mu} = \frac{du}{dx} \cdot \cos^2 \mu$$

This axial strain can be converted into axial stress by multiplying by the modulus of elasticity, *E*. The resultant axial force over the differential angled element width, *dw*, can then be calculated. The horizontal component of this force can then be distributed over a differential height, *dh*. This leads to the relationship:

$$\sigma_h = \frac{F_h}{dh} = \frac{F \cdot \cos\mu}{dw / \cos\mu} = \frac{\sigma \cdot dw}{dw} \cdot \cos^2\mu = E \cdot \varepsilon \cdot \cos^2\mu = E \cdot \frac{du}{dx} \cdot \cos^4\mu$$

This is in contrast to the traditional relationship between horizontal stress and strain:

$$\sigma = E \cdot \frac{du}{dx}$$

as will exist in the flanges of the structure. Since the moment of inertia is a representation of the contribution of the area to resisting the bending moment exerted on a cross-section, the moment of inertia contribution from the web area must be reduced by the $\cos^4\mu$ factor to represent the reduced stress exerted by the web area for a given horizontal strain. This is seen in the calculation of effective moment of inertia for the built-up section.

Shear stress can be calculated using the standard beam formula, with the exception that values are converted to axial stresses in the diagonal members. Maximum shear stress, and hence diagonal axial stress, will occur at mid-height of the section.

$$f_{tc,w} = \frac{\pm 2 \cdot V}{b_w \cdot I_e \cdot \sin 2\mu}\left(Q_{f,c} + \cos^4\mu \cdot Q_{w,c}\right)$$

where b_w is the total width of the web and $Q_{f,c}$ and $Q_{w,c}$ are static moments of area of the upper or lower flange and the upper or lower half of the web with respect to the centre of the overall section.

The shear stress at a given height, y', within a section can be derived based on force equilibrium on a differential free-body above y', as shown in Figure 3.15.

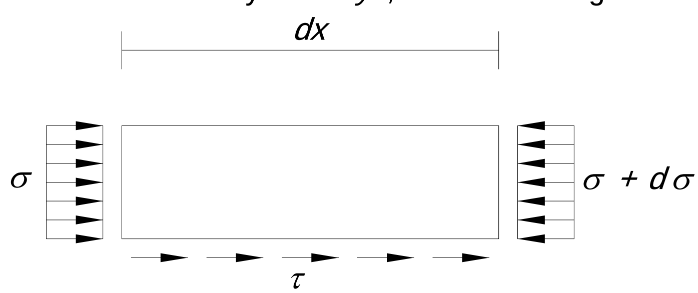


Figure 3.15 - Free-body diagram for determination of shear stress within a bending section

The resultant of the shear stress must balance the resultant from the change in axial stress over the differential length. This yields the force balance equation:

$$\tau \cdot dx \cdot b = \int (\sigma + d\sigma) dA - \int \sigma dA$$

which can be reduced to:

$$\tau = \frac{1}{dx \cdot b} \cdot \int d\sigma \cdot dA$$

where b is the width of the section at the location of the cut.

Since the equivalent axial stress generated in the web as a result of an applied moment is reduced by the factor $\cos^4\mu$, as derived in the determination of equivalent moment of inertia, the static moment of area for the web must be reduced accordingly. This leads to a final shear stress formula of:

$$\tau = \frac{dM}{dx} \cdot \frac{1}{I_e \cdot b} \cdot \left(\int_{AF} y \cdot dA + \cos^4 \mu \cdot \int_{AW} y \cdot dA \right) = \frac{V}{I_e \cdot b} \cdot \left(Q_f + \cos^4 \mu \cdot Q_w \right)$$

Finally, the horizontal shear stress must be converted into axial stress acting along the longitudinal axis of the angled web members. The resultant shear force over the horizontal length, dx, can then be calculated and represents the horizontal component of the axial force, which can be distributed over a differential width, dw. This geometry is shown in Figure 3.16.

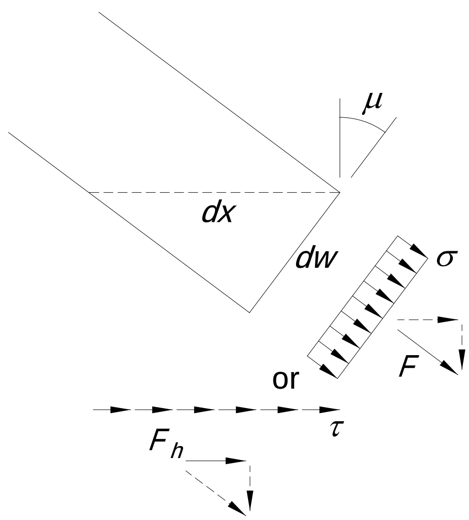


Figure 3.16 - Schematic of differential element within web of Allotey built-up girder showing stresses within angled members on a cut transverse to the member and a cut transverse to the height

The axial stress can be calculated as:

$$\sigma = \frac{F}{dw} = \frac{F_h / \cos\mu}{dx \cdot \sin\mu} = \frac{\tau \cdot dx}{dx} \cdot \frac{1}{\cos\mu \cdot \sin\mu} = \frac{V}{I_e \cdot b \cdot \cos\mu \cdot \sin\mu} \cdot \left(Q_f + \cos^4 \mu \cdot Q_w \right)$$

Using a simple trigonometric identity, and accounting for both the fact that the maximum resulting stress will occur at the centroid and that the axial stress will be either poitive of negative depending on which angled member is being assessed, the final relationship can be stated as:

$$f_{tc,w} = \frac{\pm 2 \cdot V}{b_w \cdot I_e \cdot \sin 2\mu} \left(Q_{f,c} + \cos^4 \mu \cdot Q_{w,c} \right)$$

3.3.4.2.2 - Results

Girders were designed to support the greater of H10-44 and pedestrian loading using the equations described. Permissible stress values for Group B wood from Table 3.6 were used and adjusted using a duration factor of 1.25. Results of the design are presented in Table 3.10. Further details regarding the calculations are provided in Appendix B.

Table 3.10 - Results of design of Allotey Girder timber bridge to support H10-44 loading using Group B wood

Span	m	4.0	6.0	8.0	10.0	12.0	14.0	16.0	18.3
h	mm	640	760	805	900	1060	1240	1325	1810
μ	degs	45	45	45	45	45	45	45	45
b_{lf}	mm	203	203	229	229	229	229	254	254
h_{lf}	mm	178	203	203	254	254	279	279	305
b_w	mm	102	102	102	102	102	102	102	102
d_{bl}	mm	131	128	151	127	126	135	123	115
I_e	mm^4	2.29E+09	3.86E+09	5.44E+09	8.06E+09	1.26E+10	1.97E+10	2.74E+10	6.36E+10
Q_e	mm^3	4.61E+06	6.52E+06	8.57E+06	1.17E+07	1.51E+07	2.00E+07	2.59E+07	4.28E+07
$f_{t,f}/F'_t$		0.99	0.99	0.99	0.99	0.99	1.00	1.00	0.99
$f_{t,w}/F'_t$		0.18	0.17	0.17	0.17	0.15	0.15	0.16	0.15

3.3.4.2.3 - Conclusions

Advantages:

- Efficiency: when compared with the beam, produces a greater span for a given available member size. The largest cross-section used in the 18.3 m span is the lower flange at 76 x 305 mm, significantly smaller than any of the cross-sections designated for the beam bridge
- Relatively simple fabrication: requires only aligned drilled holes and bolts; only simple tools are required
- Relatively intensive in low-skilled labour
- Primarily timber with no use of steel gusset plates for connections

Disadvantages:

- Requires relatively consistent web member dimensions to ensure fit
- Lack of detailed evaluation: bolted connection strength and design not evaluated, in particular with effect of scaling to larger sizes
- Large overall section height which needs to be below deck, with no discussion of support details
- Transport to site may be challenging with large girders if built in a workshop, as suggested by Allotey
- Low web stresses, indicating an inefficient use of material
- Details of flange member splicing not developed
- Requires large numbers of steel bolts, though these are suggested to be able to be manufactured from steel reinforcing bar, which is often readily available

3.3.4.3 - Kenyan Low-Cost Modular Timber Bridge

In a similar manner to a built-up section, a truss structure can be used to increase capacity beyond that of the single largest section available. While trusses also increase capacity by moving material away from the centre of the section, they are different from built-up sections due to their discrete nature. In a built-up section, members are connected together in many locations, creating a nearly continuous fabric. In a truss, connections are limited to specific nodal locations that are primarily located at the ends of members. This results in members having constant internal forces over their lengths, effectively dividing the truss into discrete sections. If members are furthermore relatively slender or pin-connected, all internal forces will be axial in nature.

The Kenyan low-cost modular timber bridge is a truss system intended for use in developing countries and is suggested to cost between one half and one fifth the amount of an equivalent steel of reinforced concrete bridge (Parry 1981). The system was initially proposed for rural road bridges in Kenya and a number of such bridges have been built. The system has since been refined and proposed for use in Madagascar and Central America.

The trusses of the system are made up of modules connected in series. Each module is constructed of composite timber members connected together using steel plates and bearing pins. Timber members are made up of full-depth section nailed together to create the required width. Modules are connected together using a system of male-female shear pins and plates on the top chord and steel tension ties making up the bottom chord. A schematic of a single inverted king post module is shown in Figure 3.17, and a rendering of five-module truss is shown in Figure 3.18.

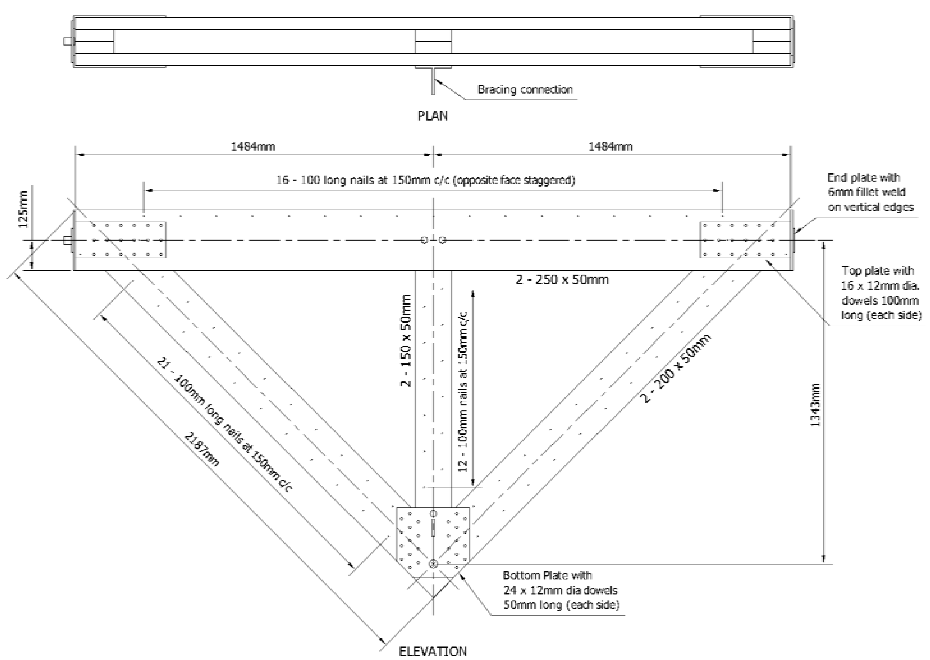


Figure 3.17 - Schematic of single module of Kenyan Low-Cost Modular Timber Bridge (Reproduced from Parry 1981)

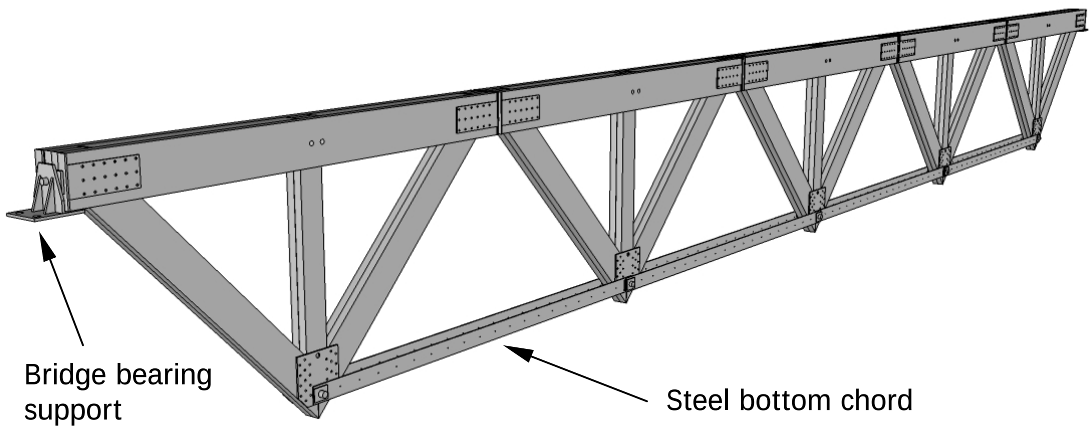


Figure 3.18 - Model of Kenyan Truss

3.3.4.3.1 - Analysis

The system can be analyzed using a simple truss analysis. The truss analysis assumes an ideal version of the system with concurrent member centre lines, perfect pinned connections, and load applied only at nodes. Member concurrency is perfectly valid in the truss with the exception of the module ends, where the diagonal members have been inset to allow room for the shear connection. This eccentricity would need to be evaluated in a more detailed analysis, but will be ignored for the sake of simplicity. Members are not perfectly pin-connected, but since no particular effort has been made to ensure a moment-connection, it is accepted practice to assume limited moment transfer. Finally, the application of loads at nodes only will be enforced through a process of distributing applied loads to the relevant nodes.

The standard algorithm for assigning distributed loads to nodes is the tributary area methods. This will be used for dead loading, pedestrian loading, and the uniform component of lane loading. In addition, point loads from wheel loads must be applied to the structure. It is common to assume a simply-supported beam between nodes and take the magnitudes of the reactions as the nodal loads. However, since each module has a continuous top chord, the reactions for a two-span beam are used instead. This analysis is also used to determine maximum moments in the top chord due to vehicle point loads. Details are provided in Appendix B.

Truss analyses are run for each possible location of the moving point loads, and the maximum forces in all members are recorded. Worst case forces from all live loading types and for each member type are then taken to be the maximum member live loading. Since it is suspected that maximum pedestrian loading was not considered as a design load in the original design, pedestrian loading and AASHTO H10-44 loading are analyzed independently. Combination live and dead load forces are finally converted to axial stress and compared with adjusted permissible stress values. In addition, the combination of bending and compression in the top chord is evaluated.

3.3.4.3.2 - Results

Based on the results of the truss analysis, recommended numbers of trusses for a given span are given in Table 3.11. These values are more conservative than those suggested by Parry, which is the result of considering maximum pedestrian loading as opposed to vehicular loading only. Example bridge cross-sections for different numbers of trusses are shown in Figure 3.19.

Table 3.11 - Recommended number of trusses for Kenyan Low-Cost Modular Timber Bridge for pedestrian or H10-44 vehicle loading

Number of Modules	Span (m)	Number of Trusses
4	12	2
5	15	4
6	18	4
7	21	6
8	24	8

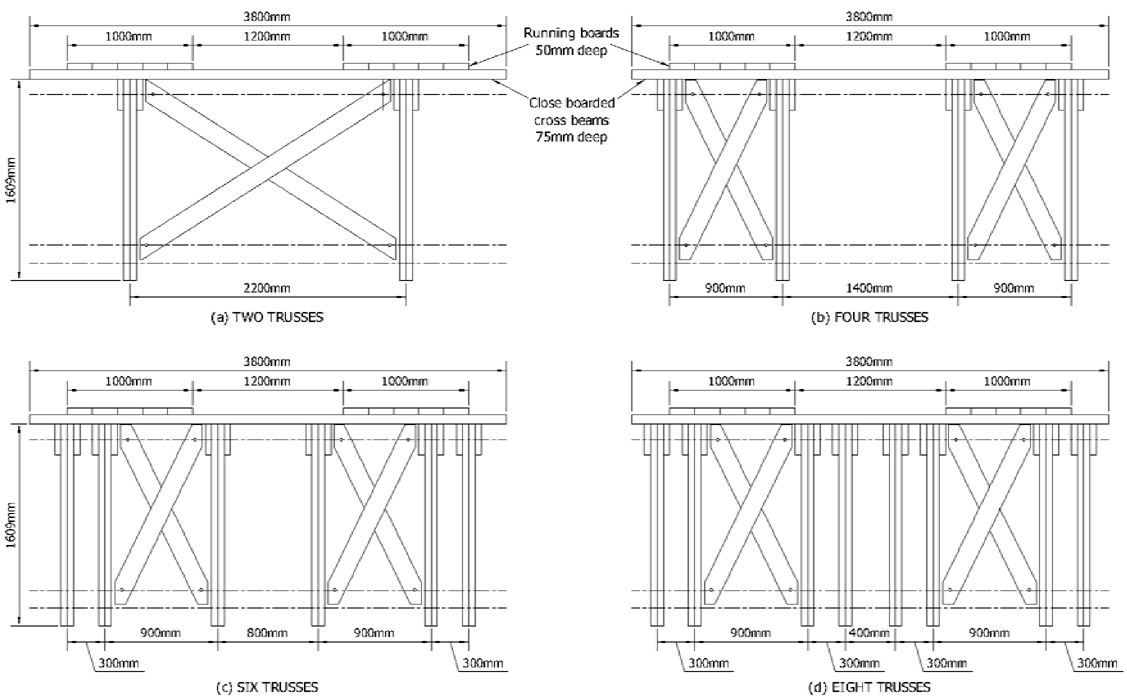


Figure 3.19 - Cross-section of Kenyan Low-Cost Modular Bridge with 2, 4, 6, or 8 trusses (Reproduced from Parry 1981)

3.3.4.3.3 - Conclusions

Advantages:
- Efficiency: greater span beyond that of largest available section. The largest cross-sections used in the truss are the members of the top chord at 50 x 300 mm, significantly smaller than any of the cross-sections designated for the beam bridge
- Modularity allows for efficient jig-based fabrication
- Modules can be fabricated in a workshop and transported separately to site

Disadvantages:
- Large amount of steel used in the form of connector plates and steel bottom chords.
- Steel workshop required for fabrication
- Strength reliant on skilled steel workmanship, particularly welding at inter-module shear connections and tension chord connections
- Discrete lengths in 3-meter increments only: may be difficult for some spans and especially problematic if replacing a bridge and reusing existing abutments.

3.3.4.4 - Comparison Between Timber Bridge Types

Having designed each bridge type for the loading and wood that are likely to be seen in the Tshumbe Diocese, it is now possible to compare between the systems and render some conclusions. One useful method of comparison between the systems relates to the efficient use of materials. For bridges, this can be a comparison of the span as a function of the amount of material needed. Figure 3.20 shows plots of span vs. volume of wood for each of the three bridge systems. Two separate curves are provided for the Kenyan bridge to account for the inclusion of the steel bottom chords. In the first curve, labeled “Steel Weight”, the weight of the steel is simply added to the weight of the trusses and converted directly into volume using the density of wood. In the second curve, labeled “Equivalent Timber Chords”, the steel bottom chords are converted into equivalent timber cross-sections based on the ratios of allowable stress.

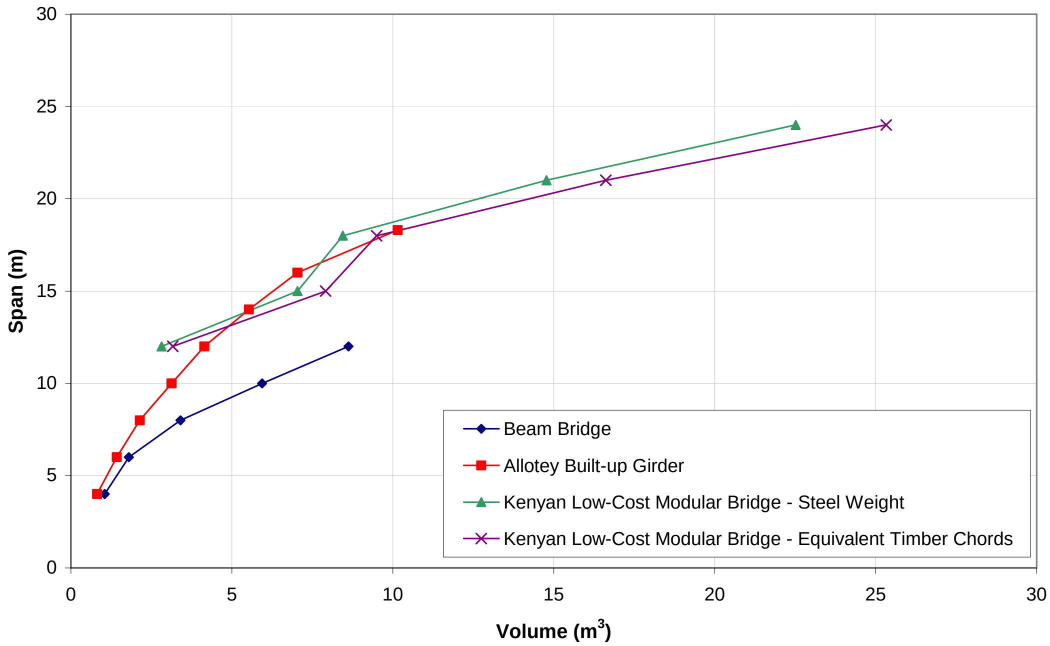


Figure 3.20 - Comparison of span as a function of volume of material for timber bridge types

The timber beam bridge is represented by the lowermost curve in Figure 3.20, offering the least span for a given volume of material. As can be seen in this curve, there is a non-linear relationship between the span and volume of material, with each successive increase in volume offering a smaller increase in span. This is a result of volume being dependent both directly on the span itself, and on the cross-sectional area of material, which will increase with span due to a need for a greater capacity. This non-linear relationship is evident in all three of the curves. At the maximum end of the timber beam bridge curve, the cross-sectional area of each beam is 0.25 x 0.575 m (0.82 x 1.89 ft). This is a relatively large cross-section for a solid beam, but is not unreasonable in terms of overall size for manipulation on site or overall depth below the deck.

The curve for the Allotey Girder starts near to the beginning of the beam bridge curve, though with slightly less material for the given span. Because of greater efficiency fo material use, the curve for the Allotey Girder separates from the beam bridge curve, offering a significantly greater span for a given volume of material. The design used for the curve includes significantly more web material than needed for adequate shear strength, which would be an area to explore to improve efficiency. At the same time, however, the amount of material shown does not account for lap splicing of the flange members, which would increase the amount of material needed.

Despite the potential increase in material, the girder is still a relatively efficient system in terms of its use of material. Its primary advantage is the ability to increase the depth of the section, unlike the Kenyan Truss, while limiting the material near the neutral axis, unlike the rectangular beams. However, the increased depth of the section is also one of the primary drawbacks of the system. The section for a 60 ft span has an overall height of 1.81 m (5.94 ft), which necessitates a large amount of clearance between the deck and the high water level, and also challenging abutments or approaches to allow the roadway to be at the same elevation as the bridge deck.

The Kenyan Bridge is represented by the final two curves shown, with efficiency similar to that of the Allotey girder and offering a much higher maximum span, along with a corresponding higher material use. The main advantage of the Kenyan Truss lies in its modularity, while the main potential disadvantage is in its abundant use of steel components. For a location that has the appropriate workshops, skilled labour, and steel availability, the truss is likely to be a good choice for use in rural road bridges.

The final Kenyan Bridge truss has an overall height of approximately 1.5m (5ft), which is not insignificant, but is less than that used for the Allotey girder at 60 ft, and can be used to support spans up to around 80 ft. Furthermore, the Kenyan truss has an abutment design that allows the truss to easily be supported while keeping the bridge deck at the same elevation as the roadway.

3.4 - Timber Bridge System for the Tshumbe Diocese

For short spans, timber beam bridges are the most appropriate choice. Beam bridges are currently being used in the Tshumbe Diocese, and there is no need or worth in changing the system to a more complicated one. If the material is available, a beam bridge is the simplest and most appropriate option.

For longer spans, a decision needs to be made between using a string of beams with mid-span supports and using a bridge system that can provide a longer functional span. This decision will be largely economic in nature. If reliable mid-span supports are possible, it may be easier and less expensive to build supports and work with simple beam bridges. In some case, reliable supports will be difficult or impossible with the technology available, in which case it will be necessary to change to a different structural system.

For the Tshumbe Diocese, neither the Allotey Girder nor the Kenyan Bridge are an ideal choice. The Allotey Girder uses material relatively efficiently, but has a problem with needing a significant amount of depth below the deck and especially at the supports. The Kenyan bridge on the other hand is a well-designed option that is largely inappropriate for the Tshumbe Diocese due to its use of significant quantities of steel. There is a need for an alternative system that accounts for these inappropriate aspects.

In discussing the application of the Kenyan Bridge, Parry suggests two covered bridge truss types, specifically the Town lattice truss and the Howe truss, which might offer alternatives for longer spans. The Town lattice truss is of particular interest since it bears a number of similarities to the Allotey Girder. One of the most significant differences is that the Town lattice truss has spaces between diagonal web members, which allows the truss to be used as a through-truss, with deck members sitting on the lower chord and traffic traveling between the trusses. This reduces the amount of structure beneath the deck, which is one of the most serious disadvantages of the Allotey Girder.

A second difference between the Town lattice truss and the Allotey Girder is the use of wooden peg connections as opposed to steel bolts. This essentially makes the Town lattice truss a fully wooden truss, which is an appealing concept as an appropriate technology for the Tsumbe Diocese, and considered to be the primary advantage over the Kenyan Bridge and the Howe Truss.

The Town lattice truss will be presented in detail in Chapter 4.

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Chapter 4 – The Town Lattice Truss: Overview

The timber bridge systems presented in Chapter 3 are all considered to have some characteristics that make them not an ideal choice for bridges that exceed the capacity of the current beam bridge technology in use in the Tshumbe Diocese. The Town lattice truss bridge is thought to potentially offer a more appropriate solution, and must be evaluated accordingly.

In this Chapter, the Town lattice truss is presented. The original development of the structural system is first framed within the broader context of the development of timber covered bridges in the United States in the early 19th century, and followed by a technical overview of the truss.

To better understand the nature of the truss, a technical study of existing bridges was performed, surveying 40 Town lattice truss bridges in the northeastern United States and recording dimensions and technical details. This data is used to describe in detail the components and layout of the Town lattice truss and to develop a set of recommended properties for use in the assessment of existing bridges and the design of new bridges.

Having described the nature of the Town lattice truss, the characteristics that make it an appropriate choice for developing countries are described and used to compare the truss with other timber bridge systems. Finally, a simple structural assessment of the existing Town lattice truss bridges is performed to investigate the validity of existing bridges as models for the design of new bridges.

4.1 - Background and History of the Town Lattice Truss

4.1.1 - A History of Timber Bridges in the United States

Many historians and researchers have documented the history of timber bridges. This document will make no attempt to present a comprehensive review of this diverse topic. However, a brief review of the general timeline of covered bridge building in the United States is important to understand the context in which the lattice truss as a bridge technology was developed, introduced, and gained popularity. Information in this review comes from a variety of sources including Allen (1957), Edwards (1959), James (1982a; 1982b), Dreicer (1993), and Pierce et al. (2005).

The first half of the 19th century saw the construction of a huge number of timber bridges in the United States. Starting in the late 18th century, large-scale timber bridges were being built to accommodate the growing population of the United States and their move away from the Atlantic coast. One of the prominent builders of the time was Timothy Palmer, who is often credited as building some of the first covered American timber bridges, with specific reference given to his Schuylkill River crossing in Philadelphia, PA, finished in 1805 or 1806 and shown in Figure 4.1. Due to the expense of the bridge, a roof and sides were added to protect the structural members. Such covered bridges were found to have greater longevity than their uncovered counterparts, and the trend of covering continued in wooden bridge construction until wood preservatives made the practice less necessary.

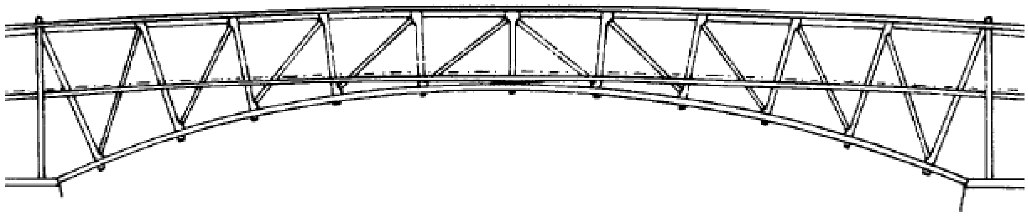

Figure 4.1 - Palmer's "Permanent Bridge" in Philadelphia, PA (James 1982b)

Louis Wernwag and Theodore Burr were two contemporaries of Timothy Palmer, and, along with Palmer, built a significant number of wooden covered bridges in the first several decades of the 19th century. All three builders focused on the timber arch as their main structural form, with the roadway following, or even sitting directly on, this curved shape in many of the early bridges. With time, more emphasis was given to a level roadway, and Burr patented an arch-truss combination that had such a roadway on February 14, 1806. The Burr truss, shown in Figure 4.2, would go on to become one of the most widely constructed covered bridge truss types.

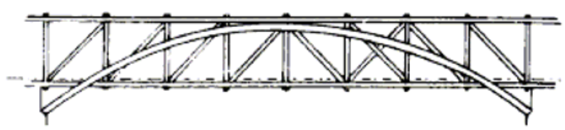

Figure 4.2 - Drawing of a Burr Truss (James 1982b)

All three gentlemen were prolific bridge builders, but Theodore Burr had more of an impact on the future of bridge building by creating and patenting a bridge system as opposed to a series of unique bridges. Timothy Palmer is also known to have received several patents for bridge improvements, but details of these have been lost, and there is no evidence of a wider acceptance or use. In comparison, while Burr continued to build bridges following his own patent suggestions, many other builders began to adopt the Burr system, leading to a much wider usage.

The Burr truss appears to be two structural systems, an arch and a truss, that could theoretically share the load on the bridge. It is, however, clear that Burr intended the arch as the main structural element, and that the truss serves a dual purpose of transferring loads into, and along, the arch and providing buckling resistance to the arch, effectively increasing its capacity for non-uniform loads. The resulting parallel chord truss also allows convenient framing for a level roadway and roof.

Burr's patent specifies an arch, as the main structural system, combined with some sort of bracing truss, the nature of which is not specified. While the early Burr trusses used a multiple king post design, this was not specifically mandated by the patent, and many of the builders who constructed Burr trusses, including Alexander Burr himself, introduced many variations on the framed beam component of the structure. It is thought by some that this may have inspired builders to experiment, since “the arch provided a physical framework into which the designer could try various structural ideas.” (Dreicer 1993)

Ithiel Town, an architect by training and vocation, was one of many who built bridges following the Burr patent. According to Allen (1957) “Town had already built the first covered bridges across the Connecticut River at Hartford, Springfield, and Northampton, using Burr’s arch type of construction. He came to the conclusion that there must be an easier way to make a wooden bridge.” While his exact motivation cannot be known, not long thereafter Town developed his “mode of construction” which abandoned the arch and focused on a framed beam using the lattice as the main underlying structure, as illustrated in Figure 4.3. Town even used aspects that distinguished his mode from the arch-based Burr truss as major selling points.

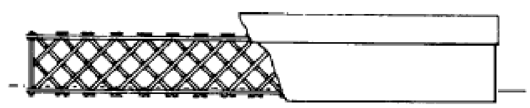

Figure 4.3 - Ithiel Town's 1820 patent design (James 1982b)

The Town lattice truss, first patented in 1820, was successful partly because of the benefits of the structural system, but also largely because of Town’s talents as a promoter.

> Except for a few test jobs in the South and a little bridge in Whitneyville in Connecticut to introduce his invention into New England, Ithiel Town built very few covered bridges himself. He was more of a promoter, the gad about salesman, who liked to pop up wherever a big new bridge was about to be built. Wining and dining the directors, he would deliver eloquent speeches in praise of his “mode,” and induce the local contractors to bid on the job and build with his plan. (Allen 1957)

Town published brochures describing and promoting his mode of construction in 1821, 1825, 1831, 1839, and 1841, which were all distributed widely. Town applied for and received a second patent in 1835, which added more detail to the original patent and expanding the use of the bridge to railroads by doubling the lattice web of the structure. This was done in response to the growing demand for railroad bridges and the potential revenue thereof.

Colonel Stephen Long was a competitor of Town’s in the 1830s, having developed a cross-braced timber bridge truss for use in railroad bridges while on detached duty from the army and working as an engineer on the Baltimore and Ohio railroad. Long’s mode of construction consisted of a cross-braced frame made entirely of timber, as shown in Figure 4.4. Wedges could be used under the end of the compression braces to control the overall shape and prestress the truss. Long’s truss incorporated an understanding of tension and compression forces and is viewed by some as the first bridge system to be based on mathematical theory. (Edwards 1959)

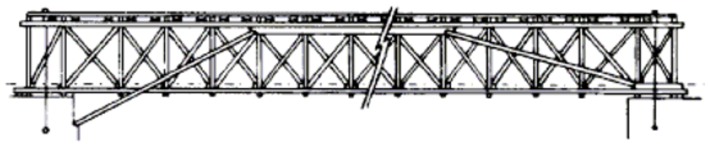

Figure 4.4 - Stephen Long's 1836 patent design (James 1982b)

Town and Long's inventions vied "for favor among the growing railroad networks, toll-bridge companies and individual town highway planners. The rival bridge promoters exchanged polite notes via the newspapers, waxing almost poetic in description of their own designs." (Allen 1957) Despite this effort, the life and popularity of the Long truss was not significant, largely due to the appearance on the scene of the Howe truss, which eclipsed both Town and Long's inventions.

William Howe's invention was essentially a variation on the Long truss, though with significant enough differences to justify a new patent and designation. The Howe truss, shown in Figure 4.5, replaced the wooden verticals of the Long truss, which required significant carpentry skill to sustain the large tension forces, with iron rods, threaded at the ends to allow for tightening during and after erection. Additionally, Howe designed patented iron seatings for the timber cross-bracing. These two changes created a robust system that could be assembled from parts in only a couple of days.

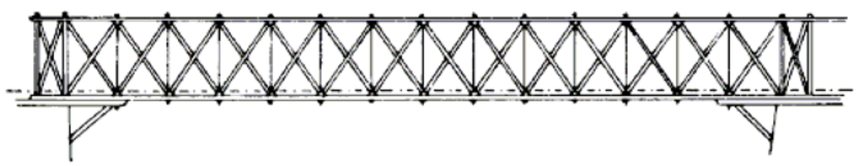

Figure 4.5 - Example of a Howe truss (James 1982b)

The Howe truss was a vast improvement on the Long truss and a completely different system from the Town truss. Whereas the Town truss focused on all wooden components and large amounts of labour to construct, the Howe truss incorporated many iron components and used these to minimize the labour cost. The Howe truss likely cost more in materials, though this would have been balanced partially by decreased labour costs, but this cost could likely be justified through the convenience and speed of assembly. The Howe truss became the more popular choice, and ultimately, more spans of this type were erected than any other type of truss.

The Howe Truss, already a composite of wood and iron, represents the last major advancement in timber bridge truss development. However, the Pratt truss offers a final coda to the subject. Designed by Caleb Pratt in 1844, the Pratt truss, shown in Figure 4.6, was another variation on the Long truss, with the diagonals replaced with iron rods. This system used more iron and was therefore more expensive than the Howe truss, and never gained popularity as a timber truss. However, as the price of iron declined, fully iron bridges became more popular and the Pratt truss and its variations became some of the most popular metal trusses.

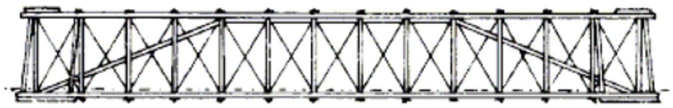

Figure 4.6 - Example of a Pratt truss (James 1982b)

By the end of the 19th century, metal trusses had eclipsed timber trusses, effectively ending the evolution of the timber truss.

4.1.2 - Background

The Town lattice truss, as mentioned above, was patented on January 28, 1820, by Ithiel Town. Responding to some of the problems with other bridge designs at the time, Town attempted to create an entirely new design and mode of construction. In many ways, he was successful, and the Town truss became one of the most popular types of bridges in the 19th century.

Figure 4.7 shows drawings from Town's first patent. The structure is simply a framed latticework. This creates a series of overlapping triangles, which make for a rigid structure. Drawings from Town's first brochure, shown in Figure 4.8, show more detail, including a second row of chords and a possible arrangement of dowels in the connections.

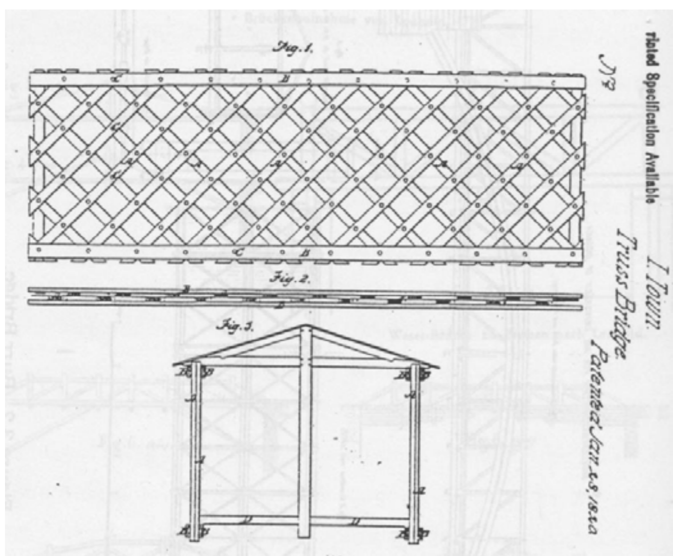

Figure 4.7 - Drawings from Town's first patent (Dreicer 1993)

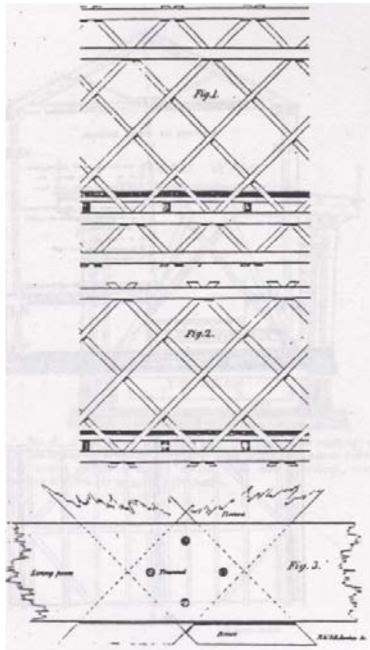

Figure 4.8 - Drawings from Town's 1821 Brochure (Dreicer 1993)

Town's second patent, in 1835, introduced the option for a second layer of lattice within the same truss, as shown in Figure 4.9. Over time, Town added more options and variations to account for different situations, but all was based on the original simple framed lattice structure.

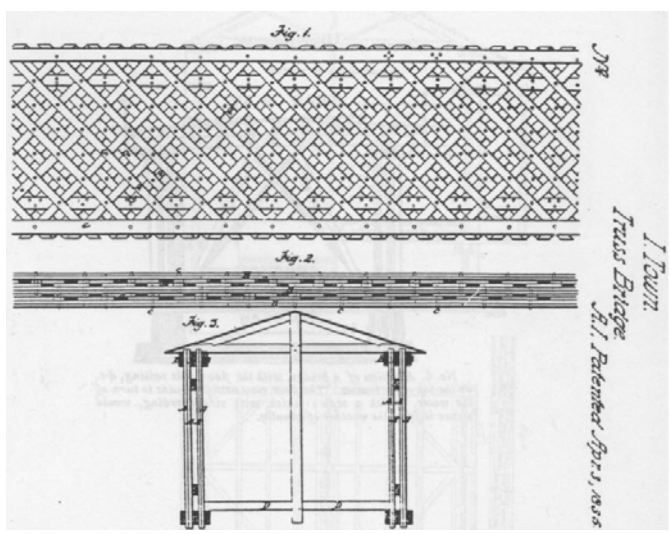

Figure 4.9 - Drawings from Town's second patent (Dreicer 1993)

A labeled schematic of the Town lattice truss as actually constructed is given in Figure 4.10, with specific components identified.

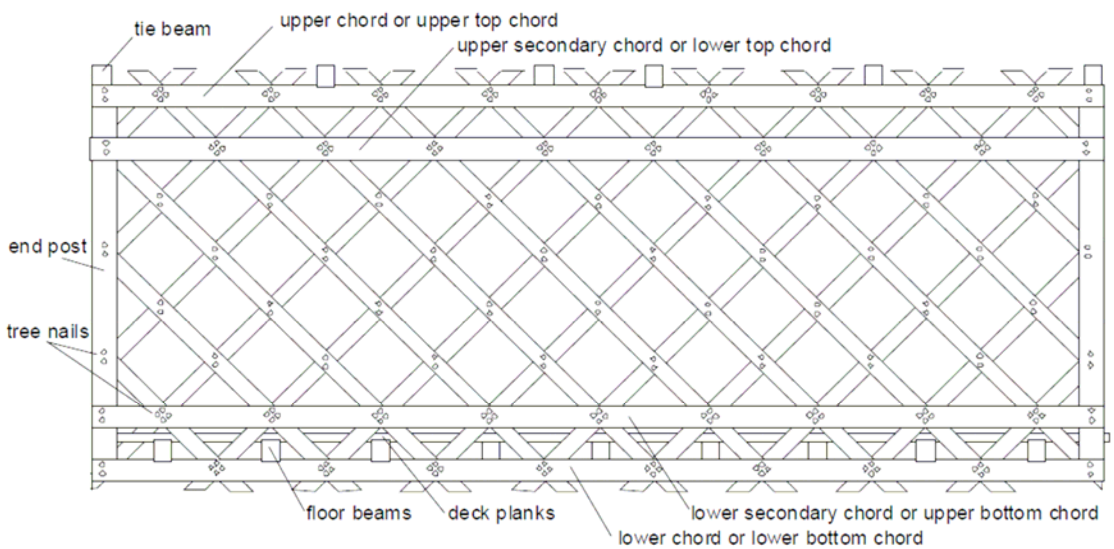

Figure 4.10 - Labeled schematic of Town lattice truss (Pierce et al. 2005)

4.2 - The Town Lattice Truss Structure

4.2.1 - Overview

The general structural function of the Town Lattice truss is typical of timber through-trusses. The bridge must be able to transfer vertical and horizontal loads exerted along the length of span to the supports at each end of the span. These loads were described in Section 3.3.2.

The two main trusses that run along each side of the bridge make up the primary structural system to support vertical loads. In addition, there are two secondary structural systems that are needed to transfer vertical loads into the main trusses. The deck system transfers vehicle and pedestrian loading onto the bottom chords of the main trusses, and the roof rafter system transfers roof loads, such as snow or rain, into the top chords of the main trusses.

In addition to supporting vertical loads, the structure must be able to resist horizontal loads, largely induced by wind. Wind loads are an increased concern on covered bridges as compared with non-covered bridges, as the covering has a tendency to increase the surface area of the bridge, thereby increasing the magnitude of the applied wind load.

The distributed wind load exerted on the side of the bridge is shared partially between a roof-level upper lateral truss, consisting of crossbeams and cross bracing, and the deck system. The load in the upper lateral truss is gradually transferred down to the deck level along the length of the bridge through evenly spaced knee braces or diagonals working in combination with lateral bending of the main trusses.

Figure 4.11 shows the components of the primary and secondary vertical load-resisting systems and horizontal load-resisting system.

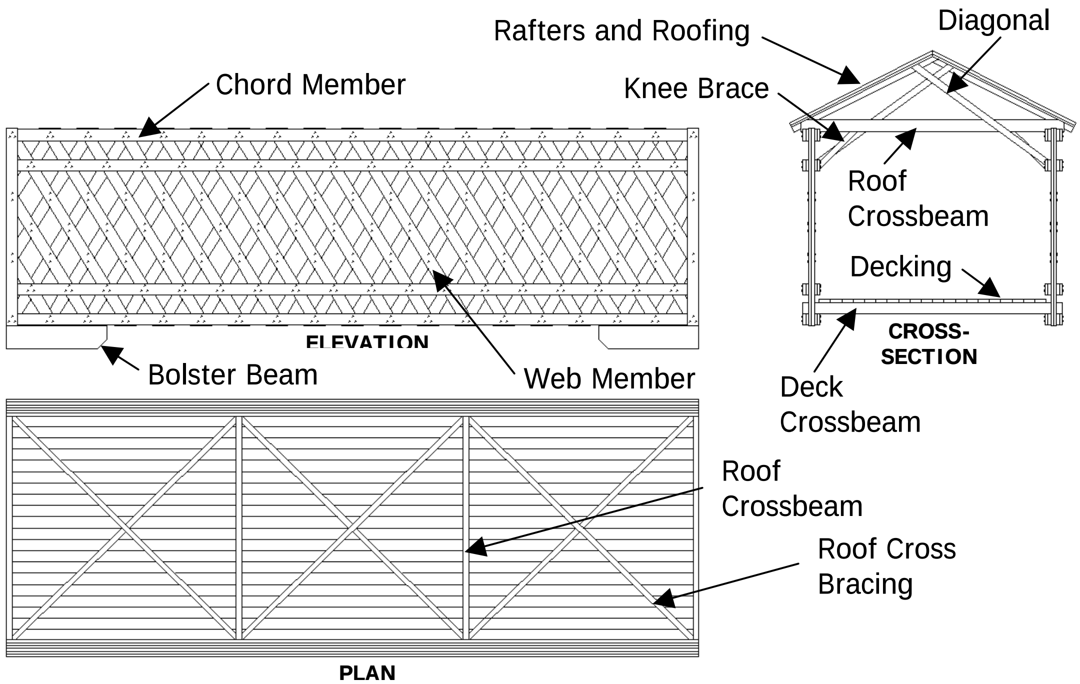

Figure 4.11 - Schematic of a Town lattice truss bridge

4.2.2 - Bridge Components

To conduct a structural analysis of a structure it is necessary to know the mechanical properties of the underlying components. The key mechanical properties for a first-order analysis of a structure are strength and stiffness. The strength and stiffness of a component will be based on both material and geometry, which are generally uncoupled and can be considered separately.

In this section, the individual components of the Town lattice truss will be described in detail and typical geometric properties will be given. The focus will be given to properties that are related to the strength and stiffness of the individual components.

Information on the components comes from a study of 40 extant Town Lattice Truss bridges located in Vermont and New Hampshire that was conducted as part of this work to gather technical details. Of the forty bridges, two were double lattice railroad bridges, one was a pony truss bridge, one was a combined arch and Town Lattice, and the remaining 36 were all single Town Lattice truss roadway bridges. Of these remaining 36 bridges, two, the Cornish-Windsor Bridge and the West Drummerston Bridge, were “timber lattice”, as opposed to the typical “plank lattice,” and two others, the Drewsville-Prentiss Bridge and the McDermott Bridge, varied somewhat from the Town patent, having a significantly sparser web than is usually seen. These four bridges will be considered as variations, and the remaining 32 bridges will be used to develop typical properties for Town lattice truss bridges.

4.2.2.1 - Main Truss

The Town lattice truss is made up a series of overlapped diagonal web members framed with longitudinal chord members. All joints are connected with wooden pegs, or trunnels, in a variety of numbers and arrangements. A schematic of a Town lattice truss structure is shown in Figure 4.12.

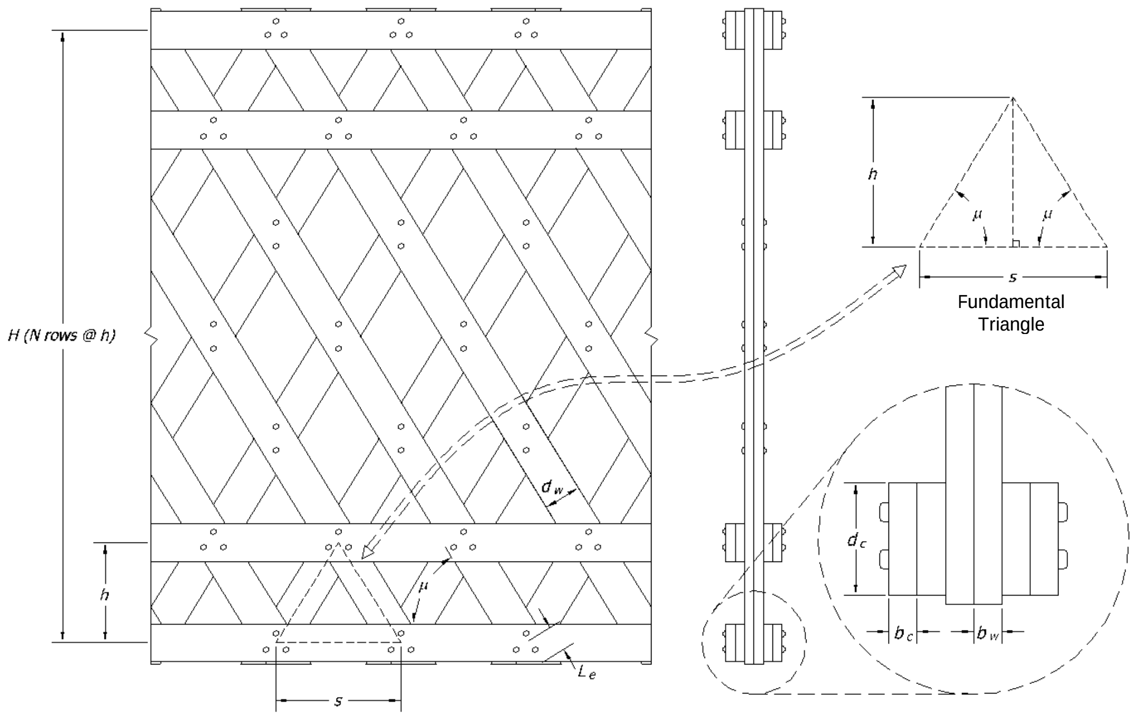

Figure 4.12 - Member layout in Town lattice truss

The various parameters that will be used in the definition of the overall geometry of the Town lattice truss, most of which are shown in Figure 4.12, are:

- H : total height of truss (on-centre from bottommost to topmost chord)
- h : vertical on-center distance between each row of connections
- μ : angle of lattice members from horizontal (note that members must have the same angle in both directions for the geometry to work properly)
- s : horizontal joint spacing
- N : number of rows of connections (7 in the Figure 4.12)
- N_c : total number of rows of chord members (4 in Figure 4.12)
- b_c and d_c : width and depth of chord member cross-section, respectively
- b_w and d_w : width and depth of web member cross-section, respectively
- L_e : extra length of web member needed beyond center of final connection (to be determined)

4.2.2.1.1 - Member Layout

The geometry of a Town Lattice truss can be defined by three properties: number of rows of joints, N, web angle, μ, and horizontal joint spacing, s. There is a direct relationship between s, h, and μ based on the fundamental triangle shape in the truss, as indicated in Figure 4.12.

$$h = \frac{s}{2} \cdot \tan \mu$$

In general, the joint-to-joint height, h, is less important than the overall height of the truss, H. There is a direct relationship between the two based on the number of rows of connections, N.

$$H = h \cdot (N - 1)$$

Combing the two equations above yields a relationship between four properties: total truss height (H), joint spacing (s), number of rows of connections (N), and the web angle (μ), which allows the calculation of total truss height based on the three defining properties.

$$H = \frac{s}{2} \cdot (N - 1) \cdot \tan \mu$$

The number of rows of joints, N, ranged from five to nine. Of the 32 included bridges, 27 had seven rows of joints, and this is considered the most common arrangement. Of the remaining bridges, one had five rows, three had six rows, and one had nine rows.

The web angle for a given bridge is calculated from measurements taken of the horizontal and vertical distance of the open diamond in the truss, using the equation

$$\mu = \arctan(vm/hm)$$

where, vm is the vertical measurement, and hm is the horizontal measurement, as indicated in Figure 4.13.

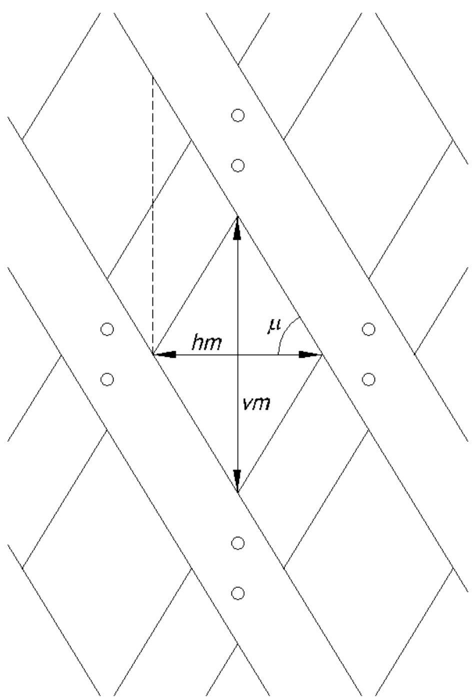

Figure 4.13 - Measurements taken for determination of web angle

Joint spacing was measured directly and truss height is calculated as described above. Information on web angle, joint spacing, and truss height is given in Table 4.1.

Table 4.1 - Information on overall truss layout

	Web Angle (deg)	Joint Spacing (in)	Truss Height (in)
Smallest	45.0	40	152
Largest	61.7	48	194
Average	51.4	46.2	170
Most Common	N/A	48	N/A

While web member angles were seen as low as 45^o and as high as 61.7^o, these values represent outliers with the bulk of the web angles occurring between about 47^o and 54^o, reflected in the average value of 51.4^o.

Joint spacings were found to consist of two groups, with most bridges having a 48" joint spacing, and all others having joint spacings on the order of six inches shorter, varying from 40" to 43.5".

Truss height varied relatively evenly between around 150" (12.5') and around 200" (16.7'). Bridge clearance height will be determined by these truss heights less the thickness of the deck system, which is typically about 18", but can reach values on the order of 28". Thus, typical clearances will range from 11' to about 15'.

4.2.2.1.2 - Member Sizes

The members in the Town lattice truss are expected to act primarily along their axis. For axial strength and stiffness, the primary geometric properties are cross-sectional area, based on depth and width, and member length between connections, which can be calculated for a given member based on the overall layout of the truss. It is possible that bending and shear will need to be considered, and the relevant cross-sectional properties can be calculated as needed from the depth and width.

As seen in Figure 4.12, each chord is made up of four parallel chord members, two on each side of the web. The members used in all of the chords are typically of equal cross-section, and will have a depth, d_c, and a thickness, b_c. The two layers of web members will also typically have members of identical cross-section, with a depth, d_w, and a thickness, b_w. Figure 4.14 shows examples of web and chord members and their connections in the upper chords of a typical Town lattice truss bridge.

Figure 4.14 - Typical chord and web members - North Hartland Twin Bridge, Hartland, VT

Member dimension information is given in Table 4.2. Each section is based on a specific number of bridges, *N*, indicated in parentheses. One bridge (Slate Bridge) was not included for web properties since it had web members of two different dimensions, with the larger being 12” x 4.5” and the smaller being 11” x 3” thick. One bridge (Chiselville Bridge) was not included for either bottom chord since they were fully concealed. A second bridge (Creamery Bridge) was not included for the lower bottom chord since large timbers (12” x 5.5”) were used for this chord only. Finally, seven bridges (Coombs Bridge, Cilleyville Bridge, Keniston Bridge, Green River Bridge, Worrall Bridge, Bartonsville Bridge, and Baltimore Bridge) were not included for the lower top chord since these bridges did not have lower top chords.

Table 4.2 - Information on member dimensions

		Depth (in)	Thickness (in)	Area (in^2)
Web (N=31)	Smallest	9	2.5	26.125
	Largest	14	4	48
	Average	10.96	3.02	33.15
	Most Common	11	3	33
Chord - Upper Top (N=32)	Smallest	9.5	2.5	26.125
	Largest	13	4	48
	Average	11.23	3.08	34.67
	Most Common	12	3	36
Chord - Lower Top (N=25)	Smallest	7.75	2.75	23.25
	Largest	12	4	48
	Average	10.78	3.13	33.88
	Most Common	12	3	36
Chord - Upper Bottom (N=31)	Smallest	9.5	2.5	26.125
	Largest	12	4	48
	Average	11.12	3.06	34.18
	Most Common	12	3	36
Chord - Lower Bottom (N=30)	Smallest	9.75	2.5	26.875
	Largest	13	4	48
	Average	11.46	3.07	35.21
	Most Common	12	3	36

As can be seen from the data, the most common web member dimensions are 11” x 3”, which yields a cross-sectional area very close to the average. The most common chord member dimensions for all chords are 12” x 3”, which yields a cross-sectional area somewhat larger than the average. This is largely due to the fact that, while many bridges have chords that are 12” deep, very few bridges have chords deeper than 12”.

4.2.2.1.3 - Chord Member Termination Patterns

In a timber truss of any reasonable span, members of sufficient length to cross the entire length are unlikely to be available. This has the greatest impact on the design of the bottom chords, which carry tension forces. A continuous structural fabric must exist to carry tension forces and therefore a mechanism must exist to transfer forces across member terminations.

In many trusses, large members are spliced together longitudinally, ideally in a way that maintains as much of the capacity of the member as possible. This requires a significant level of detail design and skilled carpentry work to function properly. As an additional concern, these splices are generally part of the critical load path, and were one to fail, it would result in an overall failure of the entire structural system.

Town proposed a different method of maintaining tension capacity in the bottom chord. Instead of providing longitudinal splices, Town built up the bottom chord out of four parallel rows of plank members and simply staggered the member terminations between the different rows. All load carried by one chord member is gradually transferred into the other parallel members through shear forces in the multiple pegged connections. An example of a section of Town lattice truss chord with staggered connections is shown in Figure 4.15.

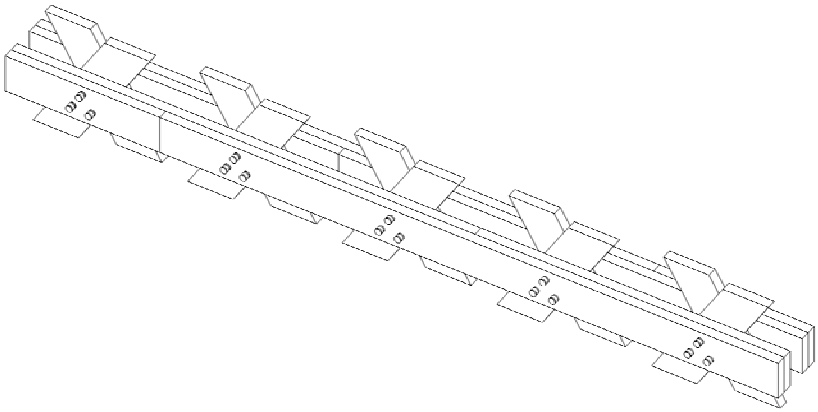

Figure 4.15 - Section of Town lattice truss chord showing staggered member terminations

There are many possible arrangements of chord terminations. If the members used in the chord are of a consistent length a repeating pattern will develop. It is common practice to position chord terminations midway between joints, resulting in member lengths that are an integer multiple of the joint spacing, *s*. The example shown in Figure 4.15 has member lengths of 4*s*, yielding a resulting pattern that is also 4*s* long.

Chord termination locations were recorded for the bridges that were a part of the study. Two or three chords were generally recorded for each bridge depending on the visibility of the chords. A numerical recording system was used in the study, and the resulting patterns were compiled. These patterns can be represented schematically, as illustrated in Figure 4.16 for the example pattern given in Figure 4.15. The full selection of patterns seen in the bridge study is shown in Figure 4.17.

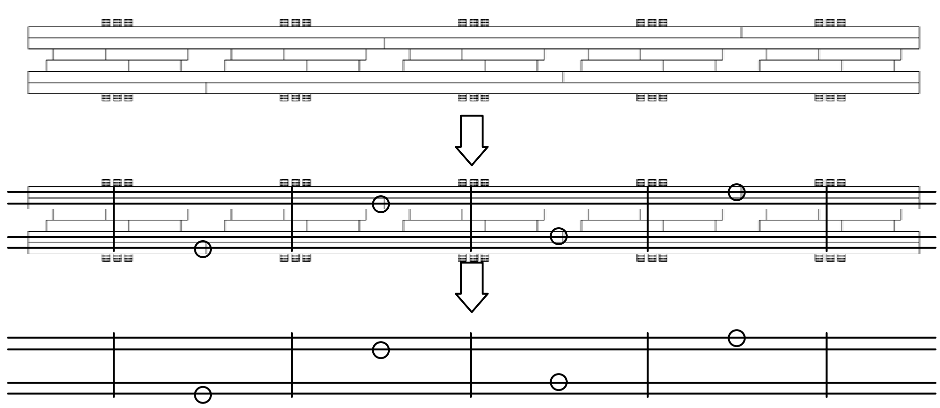

Figure 4.16 - Relationship between plan view of an example chord member termination pattern and its associated schematic diagram

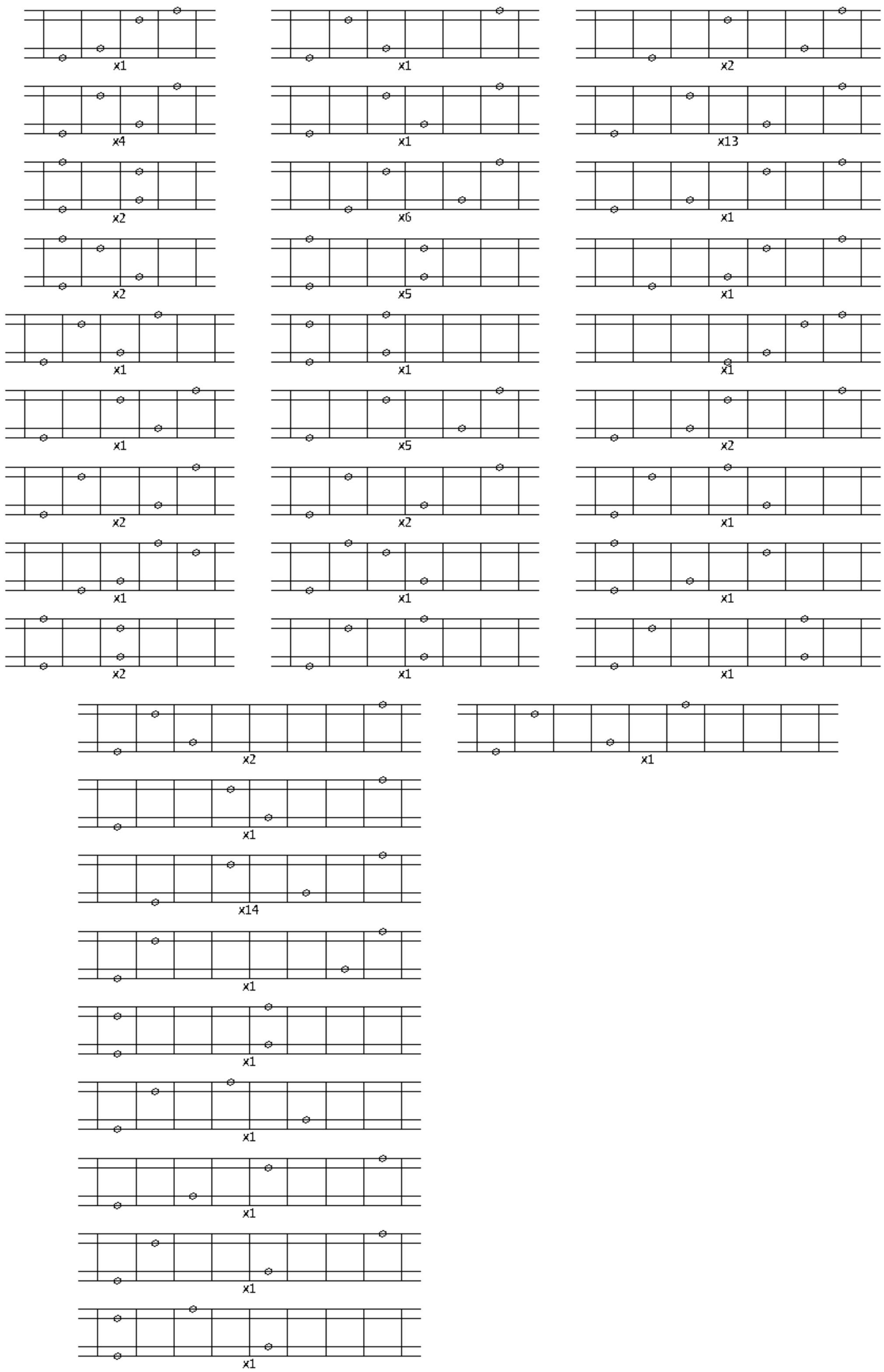

Figure 4.17 - Schematic diagrams representing set of patterns seen in use in Town lattice truss bridges

The patterns seen in the bridge study vary significantly. The most common patterns show a similar staggered arrangement of terminations that may be an attempt to spread out the breaks in adjacent rows as much as possible. But at the same time, there are patterns that seem to group the terminations close together, sometimes even placing two terminations between the same pair of connection lines. Some patterns exhibit a type of rotational symmetry while others seem to follow no obvious logic. Examples of pattern with some of these characteristics are shown in Figure 4.18. Chord termination patterns and their effect on the structural behaviour of Town lattice trusses will be examined in Chapter 5.

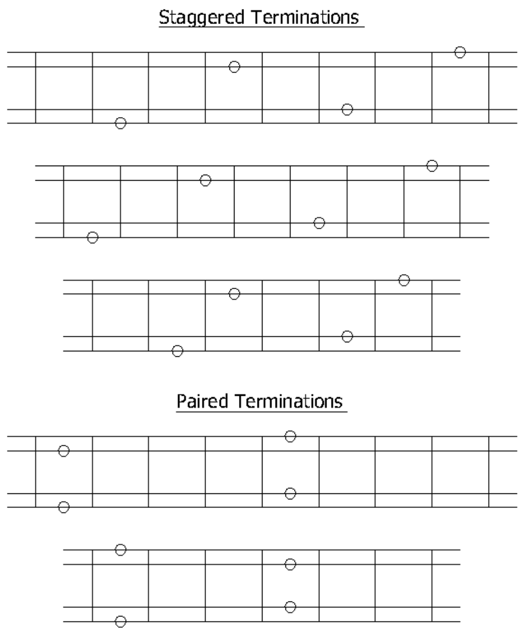

Figure 4.18 - Types of chord termination patterns identified in Town lattice truss bridges

4.2.2.1.4 - Connections

A given connection will have a translational strength and stiffness and a rotational strength and stiffness. Individual pegs each have their own strength and stiffness and the contribution of each must be summed to determine the overall connection properties. The exact contribution of each peg will depend on the arrangement of the pegs in the connection.

Peg strength and stiffness within a connection will be discussed in more detail in later sections. For now, it can be said that these properties will depend on a variety of factors including the member thickness, as presented above, and peg diameter. Peg diameters were measured and information on the results is given in Table 4.3.

Table 4.3 - Information on peg diameters

	Web Peg Diameter (in)	Chord Peg Diameter (in)
Smallest	1.5	1.5
Largest	2.25	2.25
Average	1.93	1.92
Most Common	2	2

In all but one bridge, web pegs and chord pegs had the same diameter, which is reflected in the similarity of properties but the slight difference in the average values. While 2" is the most common diameter, 1.75" is also quite common, which is reflected in the average being somewhat lower than the most common value.

The overall connection properties can be determined from the individual peg properties based on a variety of factors including the number of pegs, the pattern in which they are arranged, and the spacing within the pattern.

An individual peg will have both a shear strength, $F_{peg,max}$, and a translational shear stiffness, k_{peg}. Individual pegs are assumed to have negligible rotational strength and stiffness as a result of the potential for rotation within the holes. The translational properties will contribute to the maximum shear capacity of the connection, $F_{conn,max}$, and the translational stiffness of the connection, $k_{conn,t}$, based on a direct sum of the peg values, assuming all pegs in the connection have the same properties and that there are no group effects.

$$F_{conn,\max} = \sum_{i=1}^{N_{pegs}} F_{peg,\max}$$

$$k_{conn,t} = \sum_{i=1}^{N_{pegs}} k_{peg}$$

Peg properties will also contribute to the maximum moment capacity of the connection, $M_{conn,max}$, and the rotational stiffness of the connection, $k_{conn,r}$, based on the distance of the peg from the centre of rotation of the connection, again assuming that all pegs in the connection have the same properties and that there are no group effects.

$$M_{conn} = \sum_{i=1}^{N_{pegs}} F_{peg} \cdot r_i \quad \text{where } M_{conn} = M_{conn,\max} \text{ if at least one } F_{peg} = F_{peg,\max}$$

$$k_{conn,r} = \sum_{i=1}^{N_{pegs}} k_{peg} \cdot r_i^2$$

Web connections exhibited three different number-pattern combinations: two-peg vertical, two-peg horizontal, and three-peg triangle, all of which are shown in Figure 4.19. Of the 32 bridges, 26 had two pegs aligned vertically, three had two pegs aligned horizontally, two had three pegs arranged in a triangle, and one had random arrangements of two or three pegs.

Figure 4.19 - Photographs of various arrangements of pegs in web connections

Chord connections exhibited five different number-pattern combinations. The most common were three-peg triangles, arranged either with point up, point down, or point sideways. An example of each is shown in Figure 4.20. Additionally, there were a significant number of four-peg joints, with the pegs arranged more commonly as a diamond or less commonly as a rectangle. An example of each is shown in Figure 4.21. Finally, there were also a pair of bridges with two-peg joints, arranged either vertically or aligned with the web members, although these bridges were considered atypical for a variety of reasons. 15 of the bridges had all 3-Peg joints, 7 of the bridges had all 4-Peg joints, 8 of the bridges had a mix of 3-Peg and 4-Peg joints, and 2 of the bridges had a mix of 2-Peg and 3-Peg joints.

Figure 4.20 - Photographs of various arrangements of 3-peg chord connections

Figure 4.21 - Photographs of diamond and rectangular arrangements of 4-peg chord connections

Translational connection properties can be derived from what is presented above, since number of pegs is the only relevant factor. Rotational properties, however, would require more information about peg spacing to determine radii for the pegs in the connection. Since rotational properties for connections will not be considered in this work, radii will not be developed, however this is suggested as focus of future research on the behaviour of Town lattice trusses.

4.2.2.2 - Support Conditions

The Town lattice truss bridge is typically supported directly under the trusses at each end of the bridge. Unlike other timber trusses, which typically have only a small bearing area directly under the final node of the truss, the Town lattice truss typically has a larger bearing area, which extends over several joint spacings at the end of the truss. In some instances, the bottom chord will bear directly on a concrete or stone abutment, or on simple timber bearing blocks. However, in most cases, longitudinal bolster beams, which bear on, and extend beyond, the abutments, are used to support the trusses. An example of a bolster beam is shown in Figure 4.22.

Figure 4.22 - Bolster beam - Hall Bridge, Rockingham, VT

Bolster beams are intended to perform a combination of reducing the effective span of the bridge and distributing the end bearing force into multiple web members. The effectiveness in accomplishing either of these goals is not entirely proven. Bolster beams are expected to act primarily in bending, making moment of inertia around the horizontal axis a key geometric parameter.

Of the 27 bridges in which the support conditions could be identified, 19 were supported on bolster beams and 8 sat directly on the supports. Results of the bolster beam sizes are given in Table 4.4. Support length refers to the length of the bolster beam that is directly over supports while free length refers to the length of the bolster beam that extends beyond the supports.

Table 4.4 - Information on bolster beams

	Depth (in)	Width (in)	I_{xx} (in^4)	Support Length (ft)	Free Length (ft)
Smallest	7.5	6	527	2.5	2
Largest	16	16.5	8192	12	9
Average	11.9	9.7	3084	6.4	5.3
Most Common	12	9	6144	8	6

Bolster beam properties vary greatly, even between bridges of similar length. This is likely a result of the lack of consensus on their effectiveness. Without this, the use and properties are based solely on the preference of the individual designer.

4.2.2.3 - Truss Spacing

The spacing of the trusses is largely determined based on the logistics of providing a single clear lane for traffic. The spacing will have structural implication for the deck crossbeams required to support traffic and the roof crossbeams as part of the lateral load resisting system.

Deck width was measured for all but two of the bridges. From this value and knowledge of the member properties, the centre-to-centre spacing of the trusses can be calculated. Results for truss spacing are given in Table 4.5.

Table 4.5 - Information on truss spacing

	Truss Spacing (in)	Truss Spacing (ft)
Smallest	147	12.25
Largest	225	18.75
Average	193	16.12

There is a wide variety in truss spacing and, as mentioned, this property is largely derived from vehicle width and clearance requirements. The U.S. Forest Service Transportation Structures Handbook (Forest Service 2005) offers the following recommendations for bridge width:

> 1. Single-lane Road Bridges. Use a 14-foot (4.3 m) width as the minimum clear distance between curbs or railings for bridges, cattleguards, and other single-lane highway structures. Use widths in excess of 14 feet (4.3 m) to accommodate curve widening, off-highway vehicles, and minor deviations (up to 2 feet (0.6 m)) resulting from using standard modular structural units. Structures on single-lane roads may have the width reduced to not less than 12 feet (3.6 m) if the lesser width is consistent with the intended use. Ensure that a single-lane bridge does not create the appearance of two lanes of traffic.

The smallest widths found in the Town lattice truss bridges actually violate these recommendations by yielding a clear distance smaller than 12’. Meanwhile, the largest widths offer more clearance than necessary for a single-lane road bridge, exceeding the recommended 14’ by a significant margin. However many of these wider bridges have supplementary railing structures to reduce the clear width while also offering additional protection for the structural members.

4.2.2.4 - Roof Framing

The framing of the roof has two main components, a horizontal truss system made up of crossbeams and cross bracing designed to resist (or distribute) transverse wind loading, and diagonals or knee bracing to maintain shape and transfer load from the upper truss to the lower transverse system. Roof framing components are identified in Figure 4.23.

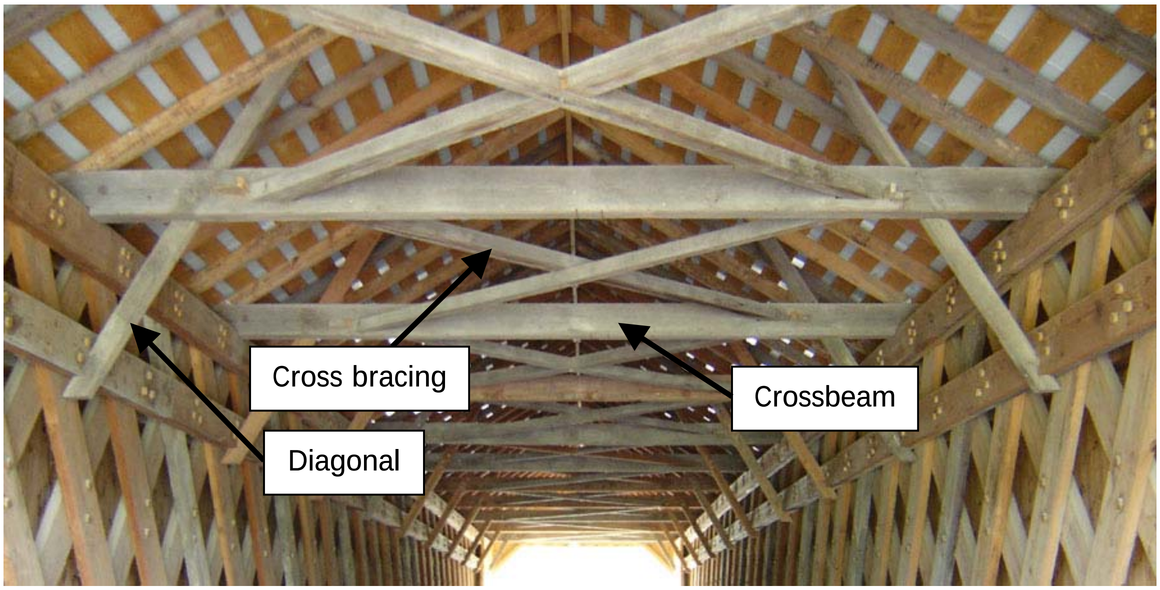

Figure 4.23 - Roof framing of Town lattice truss bridge – Sanderson Bridge, Brandon, VT

4.2.2.4.1 - Crossbeams

The crossbeams serve a dual purpose. In addition to acting as a compression strut in the horizontal truss, they also serve as a component of the vertical transfer of lateral force to the deck level. A frame is formed at the location of each crossbeam, consisting of the side trusses, the knee bracing or diagonals, and the cross beam itself. As part of this frame, the crossbeams must be able to resist the bending moment induced by the knee braces while providing adequate bending rigidity. Cross-sectional area and moment of inertia about the horizontal axis, I_{xx}, are relevant geometric properties to determine the axial and bending behaviour. As an additional consideration, crossbeams must be sized to provide enough space for cross-bracing connections.

Crossbeams are generally spaced at a multiple of joint spacing, s, primarily so they can be seated between web members. 19 bridges had crossbeams spaced at $2s$, 12 bridges had crossbeams spaced at $3s$, and one bridge had crossbeams randomly spaced. The choice of spacing will have an impact on the required dimensions of the crossbeams and the knee bracing or diagonals since it will govern the fraction of lateral load that must be supported by each lateral frame.

4.2.2.4.2 - Cross-bracing

The cross bracing is considered a component of the horizontal truss only, and due to the nature of the joinery, is generally expected to only carry compression loads. For pure compression loading, resistance to buckling, represented by the moment of inertia around each axis, is most important. Cross-bracing was found in three different arrangements, alternating K-bracing between crossbeams, single X between crossbeams, and double X between crossbeams, sometimes overlapped. Cross-bracing types are illustrated in Figure 4.24. No pattern could be distinguished in the choice of cross-bracing system.

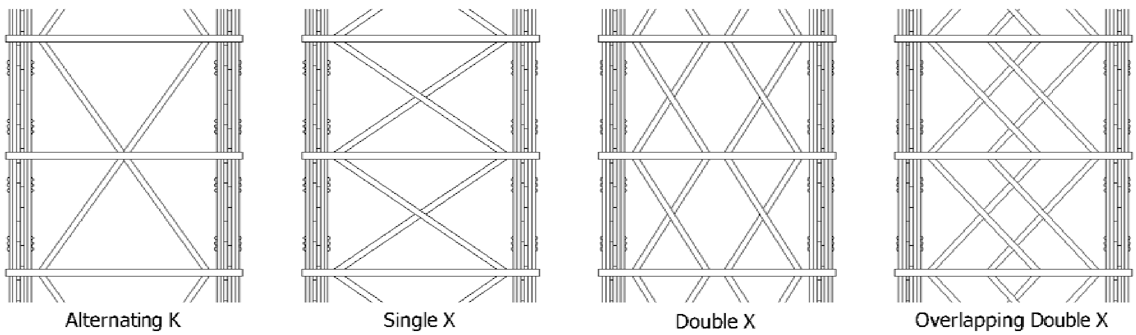

Figure 4.24 - Roof cross bracing types

There is some question as to the value of cross-bracing at the roof level in Town lattice truss bridges. If portal frames were provided at the ends of the bridge, then cross-bracing would form part of a lateral truss to transfer load into these end portals. However, since Town lattice truss bridges do not typically have solid end portals, the lateral load component at roof level must be transferred down through the Town lattice trusses to the deck lateral truss. Cross-bracing will, however, help to distribute load between knee braces in the case of a non-uniform lateral load.

4.2.2.4.3 - Lateral Bracing

The lateral bracing forms a part of the horizontal load resisting system, providing a mechanism for transfer of horizontal forces from the roof level to the deck level. The forces are then transferred through the deck system to the supports at either end of the span.

In cross-section, a covered bridge is a simple portal frame with effective pin connections at all four corners. Without some sort of diagonal bracing, the frame would be unstable and unable to support any horizontal load. Knee bracing or diagonals must be added to stabilize the frame and can act largely in compression to resist overturning, as illustrated in Figure 4.25. Diagonal members that cross and are connected to each other above the roof crossbeam will have a more complicated behaviour and may also exhibit bending.

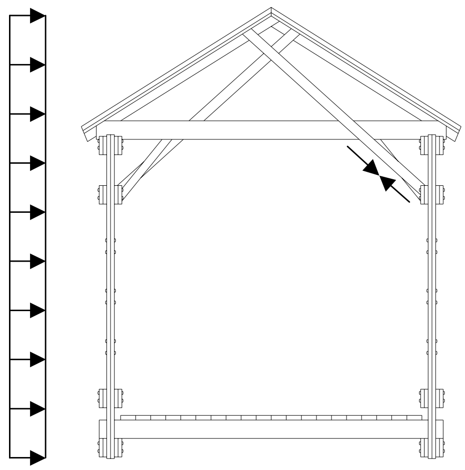

Figure 4.25 - Town lattice truss bridge cross-section showing force in lateral brace

In the studied bridges, lateral bracing varied tremendously, both in type and size. The first and most common component was main diagonals, running from the lower top chord, past and connected to the roof crossbeam, and connecting to the opposite diagonal and/or roof rafters. The second component was knee braces, running from the lower top chord and attached to the crossbeams, either underneath or on the side. The third and least common component was secondary diagonals, typically running from the upper top chord and connecting with the main diagonals near the roof peak. Two bridges had all three components, three bridges had both main and secondary diagonals, two bridges had main diagonals and knee bracing, 19 bridges had main diagonals only, four had knee bracing only, and two had no lateral bracing.

Some sort of lateral bracing system is critical to transfer horizontal wind loads to the bridge supports. Thus, the existence of knee bracing or diagonals is incredibly important. For knee bracing, members are expected to act primarily in compression can have relatively small cross-sections as a result of relatively short lengths. Diagonals are generally considered to be superior to simple knee bracing as they create a more rigid frame. However, this frame action generates bending moment in the diagonal member. As a result, diagonals must generally have greater depth than knee bracing, while still having adequate width to resist lateral buckling due to the large compression forces.

4.2.2.4.4 - Roof Angle

The angle of the roof is an important component in determining wind and snow loads on the bridge. Roof angles of bridges in the study varied relatively evenly over a 20-degree range from 22 to 42 degrees. This set of angles is representative of the northeast, although shallower roofs are commonly seen in places such as Iowa. No clear design basis is obvious for the choice of roof angle, and it is though to be primarily and aesthetic choice.

4.2.2.5 - Deck Framing

There are three major components to the deck system of a lattice truss bridge. Crossbeams are the primary structural component, transferring vehicle loads to the lower bottom chord of trusses. Load is distributed to the crossbeams through decking which is generally either a single layer laid longitudinally directly on the crossbeams, or a double layer made of longitudinal stringers and a transverse decking. Additionally, the deck system often includes cross-bracing, though the value of this has been questioned. Deck framing components are identified in Figure 4.26.

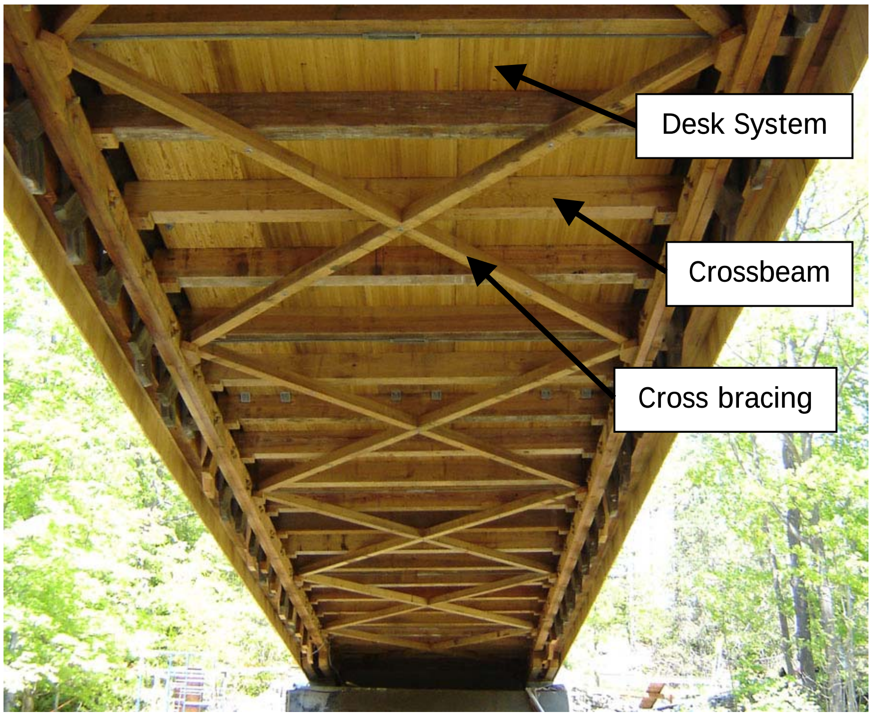

Figure 4.26 - Deck framing of Town lattice truss bridge – Downers Bridge, Weathersfield, VT

4.2.2.5.1 - Crossbeams

Loading on the deck of a bridge is transferred to the supporting trusses through the deck crossbeams. These beams must be able to withstand the shear forces and bending moments that result from a combination of distributed and point loads coming from self-weight, pedestrians, and vehicles. The key geometric properties for structural behaviour are cross-sectional area and moment of inertia around the horizontal axis.

Crossbeams are consistently spaced either at a full joint spacing, s, such that they can pass through the lattice opening with some trimming (though they do not always), or at a half joint spacing, $s/2$, such that every other crossbeam must generally sit only on the inside two chord members while the alternate members may or may not pass through the lattice opening. Examples of deck crossbeams spaced at full and half joint spacing are shown in Figure 4.27 and Figure 4.28.

Figure 4.27 - Deck crossbeams spaced at a full joint spacing, Bridge at the Green, Arlington, VT

Figure 4.28 - Deck crossbeams spaced at a half joint spacing, Corbin Bridge, Newport, NH

Of the 30 bridges that have not had their decks replaced with a steel structure, 9 had crossbeams spaced at a full joint spacing, and all of these had crossbeams passing through the lattice opening, though one was significantly cut short and only sat on three of the chord member as opposed to all four. The other 21 bridges had crossbeams spaced at a half joint spacing. Of these, 17 had members alternately passing through the lattice and sitting on only the inside chords, 3 had all members sitting only on the inside two members, and 1 had all members passing through the lattice but was only able to do this because of an exceptionally large web angle.

4.2.2.5.2 - Deck System

The primary role of the deck system is to transfer functional loading to the deck crossbeams. The structural requirements of the deck system will be largely defined by the magnitude of the expected live loads – mostly controlled by vehicle wheel loads – and the spacing of the deck crossbeams.

Deck systems were only identified for 22 of the bridges. Of these, 21 had longitudinal deck, supported directly on the crossbeams and only 1 had an intermediate layer of stringers. The majority of the deck systems were made of nail laminated vertical 2" dimensional lumber. Figure 4.27 shows a deck consisting of nail-laminated 2x6 and Figure 4.28 shows a deck consisting of 4" deep by 8" wide planks. The members of both systems run longitudinally and are supported directly on the crossbeams. The three deck systems seen in use are illustrated in Figure 4.29.

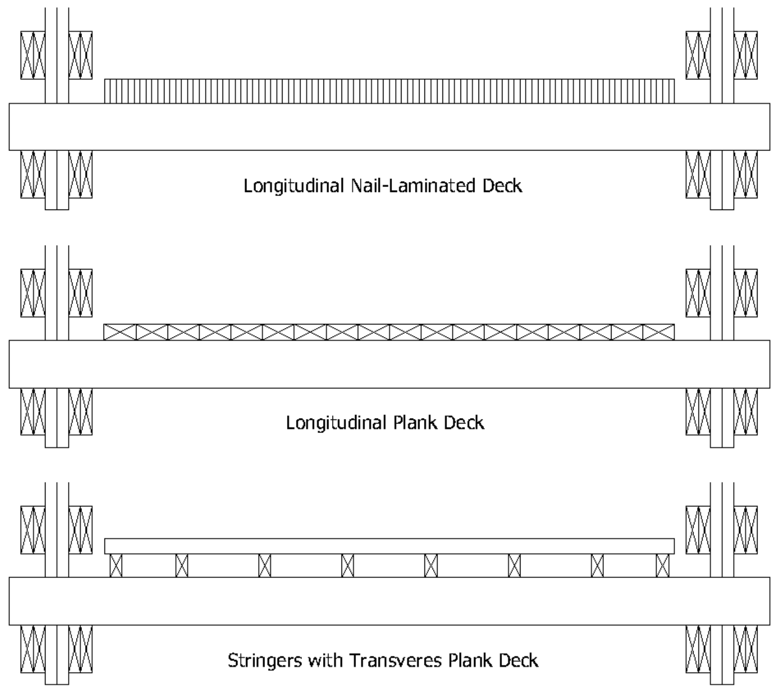

Figure 4.29 - Deck systems used in Town lattice truss

The inclusion of a stringer level is only seen rarely in Town lattice truss bridges, and is generally considered to be unnecessary. Longitudinal stringers are sometimes considered more worthwhile in other timber truss systems where there is a desire to position crossbeams close to the nodes of the truss to minimize bending moment in the truss members. In this case, the span between crossbeams will be unreasonable to span directly with decking, and a secondary layer of members is necessary. However, since the Town lattice truss has nodes at such frequent intervals, there is no value, and perhaps even harm, in attempting to spread out crossbeams along the truss. The result is crossbeams spaced close enough together that spanning with decking is entirely reasonable.

The deck system must serve two main structural roles: spanning and supporting live loads between crossbeams and providing a lateral support system for the entire bridge. The first of these is relatively easy to consider in design while the second is significantly more complicated. To provide lateral support, the deck must act as a complete lateral diaphragm. For this to be true, some form of longitudinal shear connection must be provided. This is often provided by laminating vertical deck members together or by connecting deck planks to the crossbeams.

4.2.2.5.3 - Cross bracing

24 bridges were identified as having some sort of cross bracing system underneath the deck system. These were generally wooden struts, steel rod ties, or steel cable. A further 7 bridges were identified as having no additional cross bracing. An example of deck cross bracing is visible in Figure 4.28.

The value of cross bracing at the deck level has been questioned (Pierce et al. 2005). In cases where stringers are used, as in many of the bridges original deck systems, an additional lateral bracing truss is an essential element. However, two layer deck systems, with decking sitting directly on the crossbeams, as are now predominantly used, can be detailed to provide diaphragm action that will be more effective in providing lateral strength and stiffness than cross bracing.

4.2.2.6 - Materials

Unfortunately, the wood used in the bridges could not be identified. Resources were not available for an expert visual inspection and it was not possible to take samples from the bridges in question. Thus, the wood used can only be assumed based on evidence from the literature.

In the Covered Bridge Manual published by the Federal Highway Administration (Pierce et al. 2005), the authors state “Eastern Hemlock, White Pine, and Spruce are commonly found in those bridges in the East. Douglas Fir was used in almost all western bridges. The southern covered bridges were built mostly with Southern Yellow Pine.” This is meant as a general statement about all types of covered bridges, not just Town lattice truss bridges specifically, and for this reason, no specific mention is made to the type of wood typically used for pegs.

In autumn 1995, wooden peg tests were performed at the Massachusetts Institute of Technology by McFarland-Johnson, Inc. on behalf of the Vermont Agency of Transportation in attempt to develop load capacities for the state’s 35 Town lattice truss bridges (McFarland-Johnson 1995). For these tests, Jan Lewandoski, a noted timber framer with a wealth of experience in historic timber frame structures, prepared Eastern white spruce for the members and White Oak for the pegs. These materials were considered representative of the wood used in the Town lattice truss bridges in Vermont.

In summer 2002, Engineering Report of Brown Bridge was submitted as an addendum to HAER No. VT-28, the Brown Bridge report in the Historic American Engineering

Record and became Appendix B (Conwill et al. 2002). In the report, Jan Lewandoski was consulted regarding the likely material used in the Brown Bridge and suggested "it is likely that Eastern Spruce was used for the main structural members."

Finally, in June 2002, an engineering study of the Haverhill-Bath Covered Bridge was conducted by HTA Consulting Engineers on behalf of the towns of Haverhill and Bath, NH, and the New Hampshire Department of Transportation. As part of the study, wood samples were collected from different components of the bridge and were sent to an expert for identification. Barry Goodell, PhD, Professor of Wood Science and Technology at the University of Maine, identified the lattice members as Eastern white pine and the trunnels as White Oak.

Wood properties were retrieved from the Wood Handbook for the species that are traditionally seen in the northeast and are shown in Table 4.6. Additionally, Table 4.7 gives strength design values for the species mentioned above from the National Design Specification for Wood Construction for members 3" x 12" for normal load duration and dry service conditions. For differently sized members and different loading or service conditions, appropriate factors must be applied.

Table 4.6 - Wood properties for relevant domestic woods (reproduced from Wood Handbook (Forest Products Laboratory 1999))

Species Name	Moisture Content	Specific Gravity	Modulus of Rupture (lb/in^2)	Modulus of Elasticity (lb/in^2)	Compression parallel to grain (lb/in^2)	Compression perpendicular to grain (lb/in^2)	Shear parallel to grain (lb/in^2)	Tension perpendicular to grain (lb/in^2)
Hemlock - Eastern	Green	0.38	6400	1.07E+06	3080	360	850	230
	12%	0.40	8900	1.20E+06	5410	650	1060	
Pine - Eastern White	Green	0.34	4900	0.99E+06	2440	220	680	250
	12%	0.35	8600	1.24E+06	4800	440	900	310
Spruce - White	Green	0.33	5000	1.14E+06	2350	210	640	220
	12%	0.36	9400	1.43E+06	5180	430	970	360
Oak - Northern red	Green	0.56	8300	1.35E+06	3440	610	1210	750
	12%	0.63	14300	1.82E+06	6760	1010	1780	800
Oak - While	Green	0.6	8300	1.25E+06	3560	670	1250	770
	12%	0.68	15200	1.78E+06	7440	1070	2000	800

Table 4.7 - Structural properties for relevant domestic woods (reproduced from National Design Specification for Wood Construction (AF&PA 2005))

Species and commercial grade	Design values in pounds per square inch (psi)						
	Bending F_b	Tension parallel to grain F_t	Shear parallel to grain F_v	Compression perpendicular to grain $F_{c\perp}$	Compression parallel to grain F_c	Modulus of Elasticity E	E_{min}
Eastern Softwoods							
Select Structural	1,250	575	140	335	1,200	1,200,000	440,000
No. 1	775	350	140	335	1,000	1,100,000	400,000
No. 2	575	275	140	335	825	1,100,000	400,000
No. 3	350	150	140	335	475	900,000	330,000
Stud	450	200	140	335	525	900,000	330,000
Eastern White Pine							
Select Structural	1,250	575	135	350	1,200	1,200,000	440,000
No. 1	775	350	135	350	1,000	1,100,000	400,000
No. 2	575	275	135	350	825	1,100,000	400,000
No. 3	350	150	135	350	475	900,000	330,000
Stud	450	200	135	350	525	900,000	330,000
Red Oak							
Select Structural	1,150	675	170	820	1,000	1,400,000	510,000
No. 1	825	500	170	820	825	1,300,000	470,000
No. 2	800	475	170	820	625	1,200,000	440,000
No. 3	475	275	170	820	375	1,100,000	400,000
Stud	625	375	170	820	400	1,100,000	400,000
White Oak							
Select Structural	1,200	700	220	800	1,100	1,100,000	400,000
No. 1	875	500	220	800	900	1,000,000	370,000
No. 2	850	500	220	800	700	900,000	330,000
No. 3	475	275	220	800	400	800,000	290,000
Stud	650	375	220	800	450	800,000	290,000

Table 4.6 and Table 4.7 present a wide variety of properties, and, for analysis, specific properties will need to be based on the wood used. For the analysis of existing bridges or the design of new bridges in the northeastern United States, properties from these tables should be used as appropriate. For design of bridges in the developing world, more appropriate values for tropical woods, such as those given in Table 3.6 and Table 3.7, should be used.

4.2.3 - Summary of properties in existing Town lattice truss bridges

A set of average and typical properties can be compiled based on the data presented above and further data gathered in the study of existing Town lattice truss bridges. These properties are recommended as starting values for use in the analysis and design of Town lattice truss bridges.

Member Properties
Chord: 12” deep x 3” wide
Web: 11” deep x 3” wide

Member Layout
Joint Spacing: 48” typical
Number of lines of joints: 7 typical
Web Angle: Minimum of 47.5^{o} through maximum of 54^{o} with average of 51^{o}
Truss Height: 12.5’ to 16.5’ with an average of 14.2’

Connections
Peg Diameter: 2” typical

General Layout
Truss Spacing: 16’ average

Roof Framing
Crossbeams: 10” deep x 6” wide, spaced at 3 joints spacings
Cross-bracing: 4” deep x 5” wide as a starting point
Lateral Bracing: variable
Roof Angle: site-specific, 32^{o} average
Roof: assumed 3” thick over entire roof surface for dead load calculation

Deck Framing
Crossbeams: 13.5” deep x 7” wide, spaced at half joint spacing and alternately sitting on 2 or 4 chord members
Deck System: full coverage of deck width 4” for dead load calculation
Cross-bracing: none

Support Conditions
Bolster beams: Size and length variable

Materials
Member Specific Gravity: 0.36 approximate average
Peg Specific Gravity: 0.65 approximate average

4.3 - Appropriate Characteristics of the Town Lattice Truss

As discussed in Chapter 3, timber can be an appropriate choice for construction, based solely on its functional characteristics. Beyond this, availability will be a governing factor in the appropriate choice of material. It is necessary to have access to the material itself, to the tools needed to work with the material, and labourers with the appropriate skills to work with the material. An extreme availability of any one of these factors can have a significant positive impact on a material’s appropriateness, while an extreme lack of availability can have a correspondingly negative impact.

Rural areas in timber-rich developing countries are prime locations for timber as an appropriate structural material. The availability of timber will be good, and there is the potential for a cultural familiarity with timber as a structural material. The rural location may also make access to other structural materials more tenuous and expensive, which has the compounding effect of decreasing the likelihood of tools and skills for working with these materials.

Having decided that a timber bridge may offer a solution, it is then necessary to compare the Town lattice truss to other timber bridge options. It can be compared with the timber bridge options presented in Chapter 3, including the beam bridge, the Allotey girder bridge, and the Kenyan modular bridge. It can also be compared with other through-truss covered bridges, primarily the Howe truss. The discussion and comparison herein will focus on the aspects of the Town lattice truss that render it an appropriate technology for use in developing countries.

The Town truss was a successful structural system in its time for a variety of reasons, many of which were used by Town to promote the design. In Town's 1831 and 1839 brochures (Town 1831; Town 1839), he lists 12 "advantages of constructing bridges according to [the Town lattice truss] mode". Several of these will be used to highlight characteristics of the Town lattice truss.

The appropriate characteristics of the Town lattice truss include: the lack of metal components in the truss, the use of small timbers, the use of unique wooden pegged connections, the use of a redundant and repetitive structural framework, and the ability of the truss to be used as a covered bridge.

4.3.1 - Exclusively timber truss

The first relevant characteristic of the Town lattice truss is that, more than simply being a timber truss, it is a truss constructed *solely* from timber with no metal components. Town highlights this fact:

> There is no iron work required, which at best is not safe, especially in frosty weather. This fact has, of late, been abundantly and most satisfactorily proved.

At the time the Town truss was popular, iron was still in its infancy in structural engineering. While effectively used in some structural applications, the technology was still not developed enough for use in all cases. In addition to this, iron components were extremely expensive.

While the question of the quality of iron work may have been relevant at the time, it is not as relevant in the present day. Metal components, when they are available, are generally consistent and meet with a standard for mechanical properties. However, the availability or ability to manufacture metal components may be limited in rural areas of developing countries. In such a case, a decrease in the use of metal components in the bridge can be a significant economic advantage and, in a timber-rich area, promote the increased use of local materials, a desirable quality in an appropriate technology. A lack of significant metal components distinguishes the Town lattice truss from many other bridge technologies, including the Allotey girder, the Kenyan modular bridge, and the Howe truss.

It should be noted that, while the truss itself uses no metal components, a Town lattice truss bridge would have some metal components that are not a part of the primary structural system. Nails or bolts are typically used to laminate the deck or attach it to the crossbeams and metal ties are frequently used to tie down roof crossbeams to the topmost chord of the truss.

4.3.2 - Small timbers

In addition to being built solely from timber, the Town lattice truss makes primary use of small sections. Town writes:

> Suitable timber can be easily procured and sawed at common mills, as it requires no large or long timber. Defects in timber may be discovered, and wet and dry rot prevented much more easily than could be in large timber.

While wood was generally plentiful in the heyday of the timber truss, larger sized members were beginning to be more rare, and hence more expensive, by the time Town began building bridges. The focused use of smaller planks in his design was a direct result of this lack of large timbers, and created a large economic advantage, especially compared to the arched bridges which generally needed many large and long members.

In terms of modern usage, the use of small members can be a significant advantage of the Town lattice truss design. Smaller members, both in terms of size and length, are generally easier to obtain and are also easier to transport and manipulate on site. The use of smaller members puts the Town lattice truss in a similar category to the Allotey built-up girder and the Kenyan low-cost modular timber bridge, both discussed in Chapter 3. All three systems focus on the use of smaller planks as opposed to the larger timbers needed for beam bridges and frequently used in other timber trusses.

The benefit of easier quality assessment of the wood to be used in the bridge is also extremely advantageous, particularly in developing countries where reliable visual and mechanical grading are likely to be unavailable.

4.3.3 - Connections

The main characteristic that allows the Town lattice truss to use less metal than other built-up timber trusses and girders is the wooden pegged connections; a unique system not seen in its exact form in any other application. Town discussed the advantage of these connections:

> This mode of securing the braces by so many tree-nails, gives them much more strength when they are in *tension strain*, than could be had in the common mode of securing them by means of tenons and mortises; for tenons being short and not very thick, compared with this mode, nor having so much hold of the pins or tree-nails, as in this case, will, of course, have much less power to sustain a tension or pulling strain, and it is obvious that this strain is, in many cases, equal to, and in others greater than, the thrust or pushing strain. It is also very obvious, that this pushing or thrust strain, in the mode of tenons and mortises,

> receives very little additional strength from the shoulders of the tenons, as the shrinkage and compression of the timber into which the tenon goes, is generally so much as to let the work settle, so far as to give a motion or vibration, which in time, renders them weak and insufficient.

Here, Town contrasts the wooden pegged connections of the Town lattice truss with mortise and tenon connections, a standard timber framing practice used in timber construction. Town specifically refers to the connection of braces, but as the pegged connections are used throughout the Town lattice truss, it is worthwhile to discuss all types of connections. In particular, focus will be given to tension joints, as these are recognized to be the most challenging component of timber framing (Pierce et al. 2005).

In many timber trusses, diagonal braces frame into vertical members. Many diagonals are intended to only carry compression forces, and, as a result, the connections consist of appropriately shaped bearing faces. An example of this connection method is illustrated in Figure 4.30. This type of arrangement can be found in king-post and queen-post trusses, where orientation is used to impose compression forces only, and in the Long truss, where wedges and blocks are used to prestress all diagonals, thereby ensuring members act only in compression.

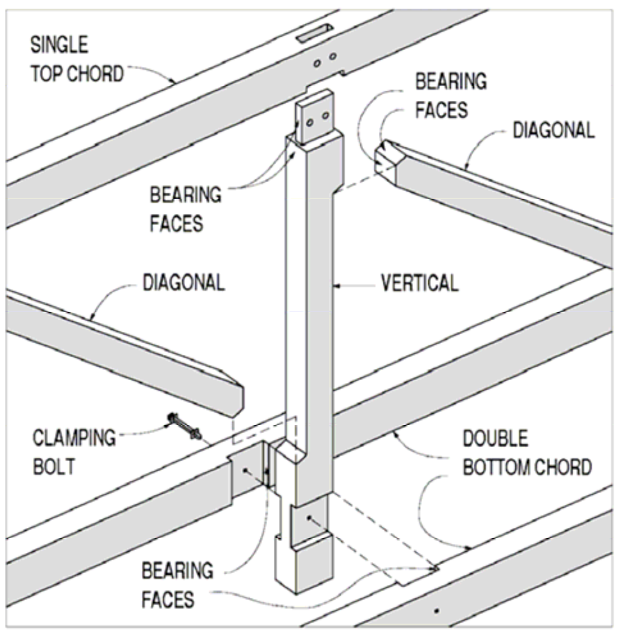

Figure 4.30 - Schematic of connections to the vertical member in a timber truss (Pierce et al. 2005)

In some case, instead of a pure bearing connection, a mortise and tenon connection can be used to connect the brace to the vertical member. This is most frequently seen in knee bracing, as illustrated in Figure 4.31. A mortise and tenon connection can allow a limited tension capacity, although such braces are generally intended to function primarily in compression. The peg or pegs that pass through the mortise and tenon are generally not intended to support load but to provide a tight fit such that load is transferred in bearing through the member. As noted by Town, the tension capacity will be very small due to the limited size of the tenon and peg. In compression, it is also possible for the compression mechanism to be affected somewhat by shrinking of the members, however this is expected to have a limited effect since the stiffness of the peg will be relatively low.

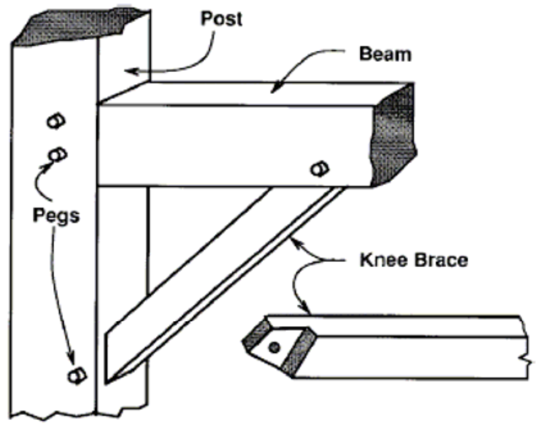

Figure 4.31 - Schematic of a knee brace connection (Bulleit et al. 1999)

Since there are no vertical members in the Town lattice truss, it is difficult to make a direct comparison. Diagonals in the Town lattice truss are connected to each other, and to the chord members, by wooden pegs that pass through all layers, as illustrated in Figure 4.32. This offers a greater potential tension carrying capacity in the diagonals through the use of full member sections, as opposed to the reduced sections of mortises and tenons, and large diameter pegs. It should be noted, however, that compression diagonals use the same peg mechanism, which may have a lower capacity than that provided by a full bearing compression diagonal.

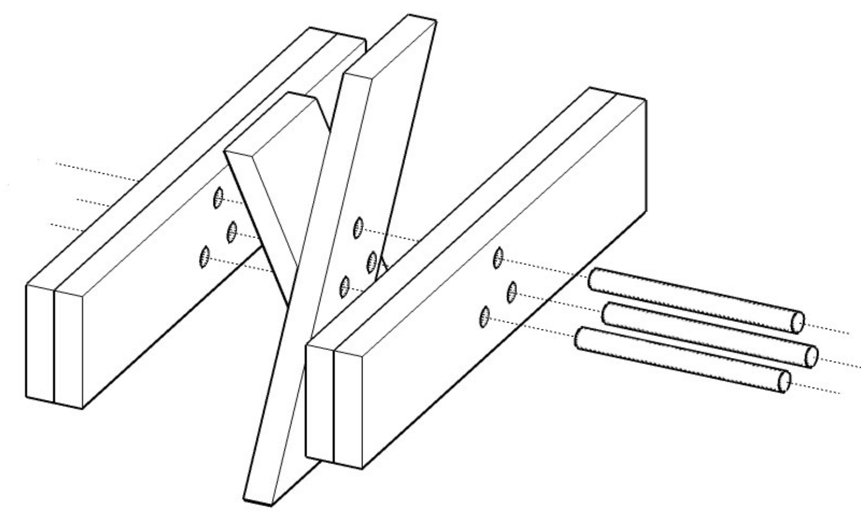

Figure 4.32 - Schematic of Town lattice truss chord connection

Perhaps more important than the tension capacity of diagonal braces is the tension capacity of the bottommost chord. The largest tension forces in a truss typically occur in the bottom chord near the centre of a span. There are a variety of connection methods that have been used to carry this tension. A simple example that is reminiscent of the mortise and tenon connections is the double-leaf lap joint illustrated in Figure 4.33. In this connection, all load is transferred through shear in the pegs. It is important to note that this and all other wooden tension connections require a reduction in area, which inherently reduces the capacity below that of the full section. The connection shown has a maximum capacity of 1/3 of the gross capacity of the section. This could be increased to 1/2 if tapered leaves were used (Pierce et al. 2005), although this entails a more challenging fabrication.

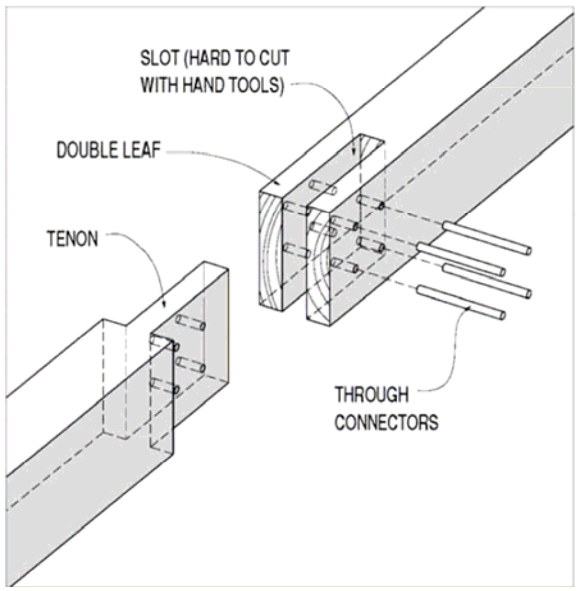

Figure 4.33 - A double-leaf lap joint, with through connectors (Pierce et al. 2005)

The design, fabrication, and final fit of traditional tension joinery is crucial to its capacity, and requires skilled carpentry work to achieve acceptable results. It is furthermore significant that such splicing in the bottom chord of a truss is typically a critical component of the load path, which will lead to structural failure if its capacity is exceeded. It is advisable to avoid introducing such detailed carpentry work into any environment where experience with structural wood is somewhat limited.

As discussed earlier, the Town lattice truss uses a different method to deal with member terminations in the chords. Instead of a single large timber member, the bottom chord of the Town lattice truss is made up of four parallel members, with terminations staggered. Individual lines of members are not spliced together longitudinally; instead load is transferred into the other lines of members through multiple pegged connections at every web member intersection, as illustrated in Figure 4.15. This shares load among a greater number of pegs than can fit into the limited area of the leaves in a double-leaf lap joint while also making no individual pegged connection critical to the load path.

Ultimately, the wooden pegged connections are one of the key features that make the Town lattice truss a unique system, and also one of the key features that make the system a potentially appropriate technology. In addition to fostering the use of local materials and local fabrication for all of the components of the truss, the connection has some functional advantages over other connections methods while requiring less skilled fabrication.

4.3.4 - Redundancy and repetition

The Town lattice truss is constructed from a large number of simple repeating components that overlap to create an extremely redundant structure. This has functional advantages as well as lending itself to more appropriate fabrication methods. Town discussed the functional advantages:

> The great number of nearly equal parts or intersections, into which the strain, occasioned by a great weight upon the bridge, is divided, is a very important advantage over any other mode; as by dividing the strain or stress into so many parts, that which falls upon any one part or joint is easily sustained by it, without either the mode of securing the joints, or the strength of the material being insufficient. Such is the advantage of this mode in this one respect alone, that no substitute in other modes, that can possibly be introduced, will ever equal it; this is equal to mathematical truth, and cannot be erroneous.

The Town truss is made up of many overlapping triangles all connected together. For any one section to deform and carry load, it must interact with other sections, effectively sharing the load. This helps to reduce the load on any individual component and allows for different load paths and force sharing if any one component is weaker or less stiff than the others.

This redundancy is in contrast to the other timber trusses of the time. The arches and trussed beams that antedated the Town truss all had a single load path, and any failure or weakness in that load path would lead to overall failure. This is especially relevant for the connections, which needed to be designed and fabricated by skilled craftsmen to ensure an adequate capacity for load.

Thus, this advantage of the Town truss is valid to a certain extent. A redundant structure of less strong components can be stronger overall than a non-redundant system made of stronger components. However, beyond this point, Town's promoter hat begins to show, and his assertions start to come into question. Town essentially states that the Truss can never fail and that it is superior to any other possible timber construction that could ever be designed. This shows a level of confidence that, while somewhat admirable, is unreasonable.

While it is true that the load does get shared between many components, if the components are all designed inadequately to support their share of the overall load, there will still be a failure. And in fact, constructing a Town truss according to the patent specifications can even lead to the entire axial tension in the bottom chord being carried by only three or four connections, which is a far cry for the multitude of sharing components implied in the description above.

Redundancy, based on the repetition of simple components to create a strong whole, is still viewed as a significant benefit of the Town truss system, however it is less related to the functional superiority of the final product than to its relation to characteristics of an appropriate technology.

An appropriate technology should provide the minimum level of required function while not wasting valuable resources exceeding it. Thus, a Town lattice truss bridge should be designed to carry a particular loading and must be compared with other bridge systems that are designed to carry the same loading. If the truss can carry loads significantly above those for which it is intended, it is over-designed and should be adjusted accordingly. The ability of the truss to be designed to carry greater loading is useful to increase the range of projects for which the truss may be appropriate. However, it is expected that most of the applications of the truss will not be at its maximum possible useful span, and this should not be a key factor in this discussion.

Redundancy has several aspects that are considered advantages. First, as opposed to increasing the maximum capacity of the structure, redundancy offers an alternate load path for the same capacity. If there is some deterioration in the structure, redundancy implies that the truss may still remain functional even if a component on the original load path can no longer support its design load. This increases the sustainability of the structure by increasing longevity and decreasing the reliance on perfect maintenance.

Second, in a similar manner to how it can help mitigate deterioration, redundancy can help mitigate small flaws and discrepancies in the original fabrication of the truss. Because there are multiple possible load paths, if a particular component is not fabricated perfectly, capacity can still be achieved. This reduces the level of skill required to fabricate the truss, increasing the potential local labour pool.

Finally, the repetitive structural framework that creates the redundancy lends itself well to fabrication with unskilled labour. Members and connections are all identical and simple to construct. With a small amount of training, a labourer will have the capacity to assemble all of the components of the truss. Despite this simplicity of construction, the redundant nature of the final product yields a truss that is both stiff and strong.

4.3.5 - Covering the bridge

The final characteristic of the Town lattice that is considered advantageous as an appropriate technology is that it is a through-truss. Having a structure that extends above the deck allows for the possibility of adding a roof and covering to protect the structural members. Town discussed this advantage:

> The side-trusses serve as a frame to cover upon, and thereby save any extra weight of timber, except the covering itself. And the importance and economy of covering bridges from the weather, is too well understood to need recommendation, after the experience which this country has already had. The

> objection, that the covering is an exposure of this bridge to wind, is not correct, nor does experience show it.

There is little relevance in this point in terms of a comparison with other timber through-trusses, all of which yield a vertical structure on which a covering can be mounted. The Town truss may have a denser structure than other through truss types, which could reduce the material needed for covering somewhat, but not to a significant degree. What is most interesting about Town's point is the clear confidence that timber bridges must be covered. By the time the brochures were written, there had been 20-30 years of experience with covering bridges, yielding an ability to compare the weathering and longevity of covered bridge with those that were not covered. Clearly, experience showed that the covering had a significant impact on the longevity of the timber bridges of the time.

This, then, is a potentially significant difference between the Town lattice truss, or any other through-truss, and a more traditional substructure. Covering a bridge was a key factor in the longevity of a wooden structure, and only ceased to be so when wood preservation treatments became standard. Longevity is an important component of sustainability, one of the required characteristics of an appropriate technology.

Effective preservation treatments require chemicals and processes that can be both expensive and potentially harmful to the environment and people if not handled properly. For these reasons, it will often not be feasible or desirable to use preservative treatments for wood structures in rural areas of developing countries. Covering a bridge may offer an alternative method to protect and preserve the key structural elements of the bridge. Maintenance will still be required to repair or replace components of the covering system, but the structural fabric, composed of the most important, most expensive, and most difficult to replace elements will have a significantly longer life.

4.3.6 - Summary of appropriate characteristics

All of the elements above increase the potential appropriateness of the Town lattice truss when compared with other timber bridge systems. However, many of these characteristics have more of an impact on the desirable elements of an appropriate technology than on the required elements. A number of the characteristics will have an impact on the economic feasibility of the project, reducing the cost of materials and skilled labour, but these will be outweighed if the overall function of the bridge is not at least equivalent to that of the other timber bridge systems. In order to assess the functionality of the Town lattice truss as a bridge system, it is necessary to have an ability to design the truss.

In the following section, the existing Town lattice truss bridges included in the bridge study will be analyzed in a simple manner to asses if the existing designs should be recreated in new bridges or if a new design methodology needs to be developed.

4.4 - The functionality of the Town lattice truss

In order to build Town lattice truss bridges in developing countries, it is necessary to have simple design rules to follow or to use to develop standard sets of dimensions and parameters. Analysis of existing Town lattice trusses has been conducted by assuming equivalent plate girder properties or performing a finite element analysis, but while these allow for the assessment of existing trusses, they provide very little insight into the general behaviour and rules for design of the bridges.

While it is clear from Town's original patent description and subsequent brochures that he had a certain understanding of structural behaviour, mathematical tools for determining stresses were not yet developed when he first patented his truss. In fact, as mentioned above, the Long truss, first patented in the 1830s, is thought by some to be the first timber truss based on mathematical theory. Furthermore, Whipple Squire's *A Work on Bridge Building*, published in 1847, is referred to as "the first significant attempt to supply a theoretical means for calculating stresses in place of the rule-of-thumb methods then in general practice." (Encyclopedia Britannica Online 2008)

If there was a lack of structural and mathematical basis in the origin the Town lattice truss, it does not invalidate the system, which still has many potential advantages as detailed above. It does, however, mean that behaviour will need to be investigated to develop reasonable modern rules that respect the original intention while ensuring an understanding of engineering mechanics is incorporated into the design.

A first step in developing design methods is to assess if existing bridges offer reasonable models for the construction of new bridges. Structural parameters can be evaluated for existing bridges and assessed to determine if they follow a reasonable design methodology. For example, an indicator of moment capacity would be expected to increase as span is increased.

A simple plate girder model is often assumed for the Town lattice truss in bending, treating the chord members as flanges connected together with an equivalent solid web of diagonals. No effort is made to account for chord termination patterns or the unique nature of the connections and lattice. Despite this, the model is commonly used and offers a simple representative design model for estimating moment capacity.

The plate-girder model assumes the Town lattice truss behaves as a bending beam with an equivalent continuous cross-section. In a bending beam, the maximum moment capacity is based on the maximum allowable stress and the section modulus, a geometric property of the cross-section.

$$M_{\max} = \sigma_{\max} \cdot S$$

where

$$S = \frac{I}{c}$$

where c is distance from the neutral axis to the extreme fiber and I is the moment of inertia of the section, based only on the chords and calculated as:

$$I = \sum A_c \cdot d_c^{\,2}$$

where A_c is the area of a chord and d_c is the distance of the chord from the neutral axis.

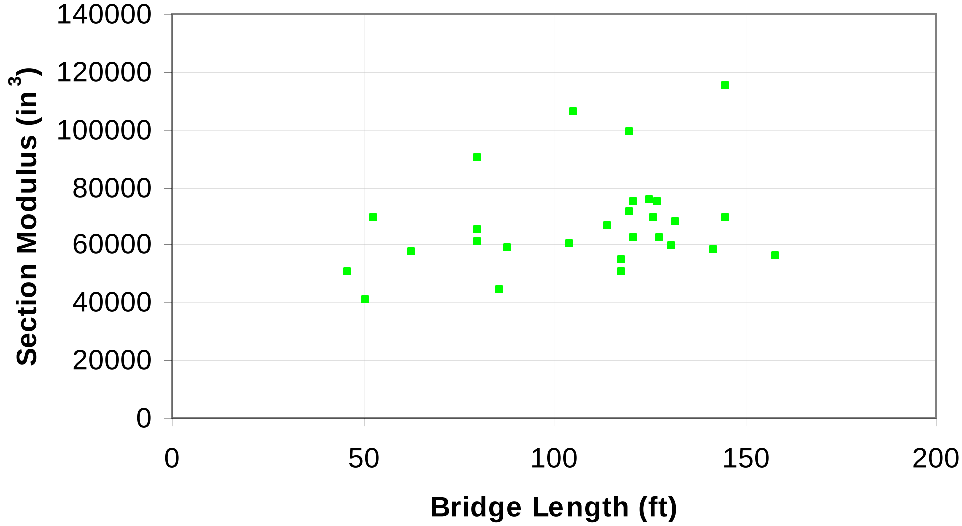

Figure 4.34 - Plot of section modulus as a function of bridge length for studied Town lattice truss bridges

Based on the plot shown in Figure 4.34, there is no obvious correlation between the section moduli existing bridges and bridge length. Many of the bridges of shorter span have significantly higher strength than other bridges of significantly higher span. In fact, the third shortest bridge has an estimated moment capacity greater than that for the longest bridge in the study.

These results suggest that the existing Town lattice truss bridges do not offer reasonable examples to be used in the design and construction of new bridges. A more detailed structural analysis approach is required to develop a new design methodology.

4.5 - Summary

The Town lattice truss is a unique structural system, which was successful in its heyday and is a potential appropriate technology for new timber bridges in developing countries. The structural system has been studied and described, and recommended properties have been given for use in the assessment of current bridges and the development of a new design methodology.

The Town lattice truss has a number of specific characteristics that help make it an appropriate choice in comparison with other timber bridge systems. A lack of metal components, the use of small timbers, the use of unique wooden pegged connections, a redundant and repetitive structural framework, and the ability to be used as a covered bridge all offer significant advantages for use in the developing world.

The listed appropriate characteristics are secondary to adequate functionality, which must still be assessed. Data collected from existing Town lattice truss bridges indicate that they do not offer models that should be copied for use in designing new bridges. To assess the functionality of the Town lattice truss as a structural system in comparison with other timber bridge system, a design methodology is needed. Such a methodology must be based on an understanding of the mechanics and behaviour of the components of the Town lattice truss.

The following chapters will investigate the behaviour of the Town lattice truss components and use the resulting knowledge to develop a simple design methodology. Chapter 5 will focus on the unique pegged connections of the Town lattice truss, Chapter 6 will focus on the behaviour of the chords in the Town lattice truss and the effect of chord termination patterns on this behaviour, and Chapter 7 will combine these elements to develop a design methodology and use the results to compare the functionality of the Town lattice truss with other timber bridges.

4.6 - References

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Chapter 5 – The Town Lattice Truss: Connections

The primary objective of structural engineering is to ensure that structures have the capacity to resist the loads to which they will be subjected throughout their lives. The overall capacity of a structure will depend on the strength of its components and, in some indeterminate structural systems, the stiffness of its components.

The Town lattice truss has a unique connection mechanism, which is one of the factors that contribute to the appropriateness of the structure. The wooden pegged connections used in the Town lattice truss reduce the reliance on metal components and increase the use of local materials. In addition, the simple assembly method of these connections reduces the need for skilled carpentry work and facilitates the use of local unskilled labour. However, while contributing greatly to the appropriateness of the structure, these unique connections have been little studied and their behaviour is not well understood.

The Town lattice truss has two types of wooden peg connections, one for the web and one for the chord. Web connections are single shear connections that typically have two pegs attaching two web members at a truss specific angle. Chord connections typically have three or four pegs connecting six members, with four members parallel and two members at truss specific angles. Examples of each type of connection are shown in Figure 5.1.

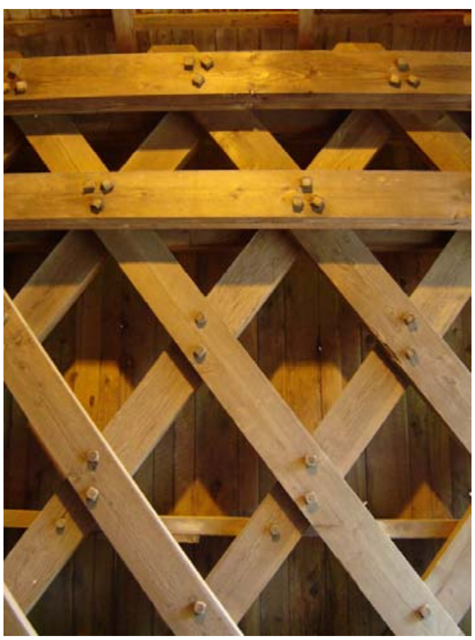
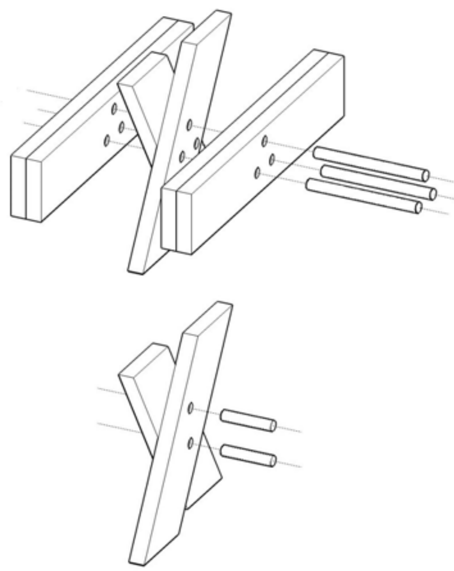



Figure 5.1 - Town lattice truss connections

This chapter addresses the strength and the stiffness of the chord connections only. Chord connections are considered to be more important than web connections for the overall structural behaviour of the truss, since they will contribute to the overall moment capacity. Web members are thought to only have a minimal impact on the overall behaviour of the truss.

Equations for the design strength of chord connections are developed based on models and experimental data from a variety of sources. Estimation equations for stiffness are also developed based on an analytical model that is assessed using experimental results from the literature.

5.1 - Connection Strength

The objective of this strength analysis is to develop an understanding of the behaviour of wooden pegged connections under yield conditions. The most important outcome of this analysis will be a prediction procedure to generate design strength values for the connections in a Town lattice truss, to be used in further analysis of larger components of the structure.

Literature on Town lattice truss connections is relatively limited. McFarland-Johnson (1995) offers some strength-related results, both for overall joints and for joint components, however due to a combination of a limited number of tests and issues in testing procedures, they are not complete and more work is needed.

Other work in wooden pegged connections has focused almost exclusively on mortise and tenon connections in traditional timber construction (McFarland-Johnson 1995; Schmidt and Mackay 1997; Schmidt and Daniels 1999; Sandberg et al. 2000; Burnett et al. 2003; Miller and Schmidt 2004). The double-shear nature of these connections and the fact that peg diameters are typically smaller than those used in the Town lattice truss make this research not directly applicable, but similar enough that much of the work can be used to extrapolate prediction rules.

Finally, the industry standards for the design of joints for wooden members with metal dowel connectors (AF&PA 2005) can be used as a reasonable starting point to develop rules for wooden peg connections.

5.1.1 - Literature

In determining connection strength, there are generally two major aspects to consider. The first is to determine minimum maximum joint strength. The second is to establish rules that determine when the full strength will be achieved (e.g. geometric restrictions) or rules that reduce the full strength to a lower value if the full strength cannot be achieved.

The accepted current methodology for determining the strength of doweled connections in timber is the use of lateral yield equations, as required by the National Design Specification for Wood Construction (AF&PA 2005), henceforth referred to as NDS. Lateral yield equations are based on the concept that joints can fail in a variety of ways, each of which can have the yield force defined by an equation based on geometric and material properties. The yield force of the weakest failure mechanism will then correspond to the force at which the overall joint will yield.

NDS only provides equations for metal dowels, and a number of researchers have made efforts to adjust these equations for use with wooden dowels (Brungraber 1992; McFarland-Johnson 1995; Church and Tew 1997; Schmidt and Mackay 1997; Schmidt and Daniels 1999; Sandberg et al. 2000; Burnett et al. 2003; Miller and Schmidt 2004). The Standard for Design of Timber Frame Structures (TFEC 2007) includes a suggested application of these research results for timber frame structures. Results of these works will be presented, with specific reference to their application for connections in the Town lattice truss.

Potential yield modes are first identified and then equations are developed to define the yield force of each mode based on general geometric and material properties in the connection.

5.1.1.1 - Yield Modes

Modes are typically illustrated using a schematic representation of a cut through the centre of a peg as it passes through multiple members. A pegged mortise and tenon connection is shown in Figure 5.2, with cut sections both before and after deformation.

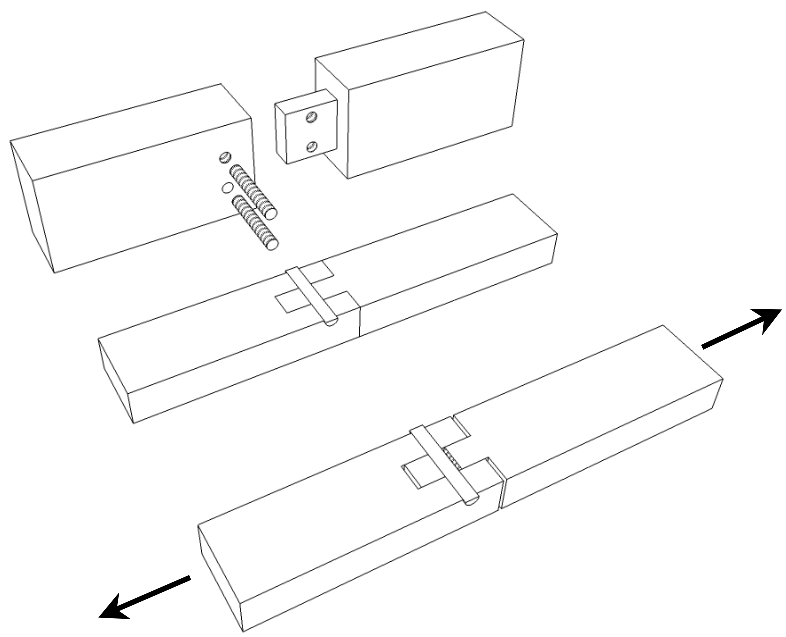
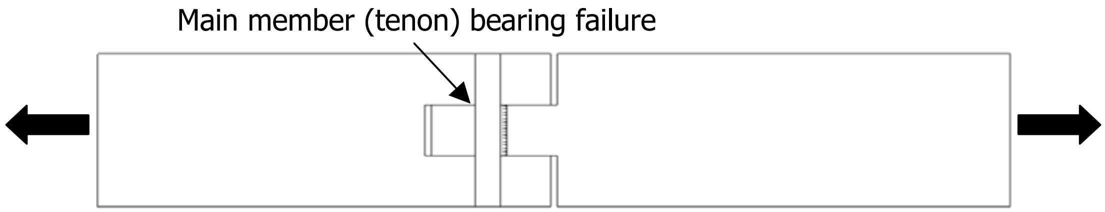
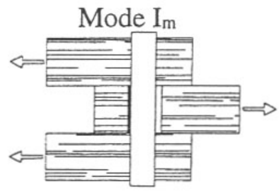


Figure 5.2 - Typical mortise and tenon connection with section cuts through an undeformed and a deformed connection

If it was of significant enough magnitude, the deformation shown in Figure 5.2 might indicate a failure in crushing of the main member. In this case, the failure mode might be represented as shown in Figure 5.3.



(a)



(b)

Figure 5.3 - Crushing of main member (mode Im) shown in (a) section view of a mortise and tenon connection and (b) schematic illustration of failure mode (Schmidt and Daniels 1999)

NDS proposes yield modes for both single shear and double shear connections with metal dowels. The proposed yield modes are shown in Figure 5.4. The mode identification scheme used in NDS is followed and expanded for use in timber dowel connections. Modes $\mathbf{I_m}$ and $\mathbf{I_s}$ refer to bearing failure in the main (larger for single shear or center for double shear) member and side (smaller for single shear or outside for double shear) member, respectively. Mode **II** represents a member bearing failure resulting in rotation of the peg within a single shear connection. Mode $\mathbf{III_m}$ and $\mathbf{III_s}$ represent bending failures in the dowel, with $\mathbf{III_m}$ including a bearing failure in the main member due to peg rotation (only possible with a single shear connection) and $\mathbf{III_s}$ including a bearing failure in the side member due to peg rotation. Finally, **IV** represents a bending failure in the dowel with hinges forming within the thickness of each member and exhibiting limited member bearing failure.

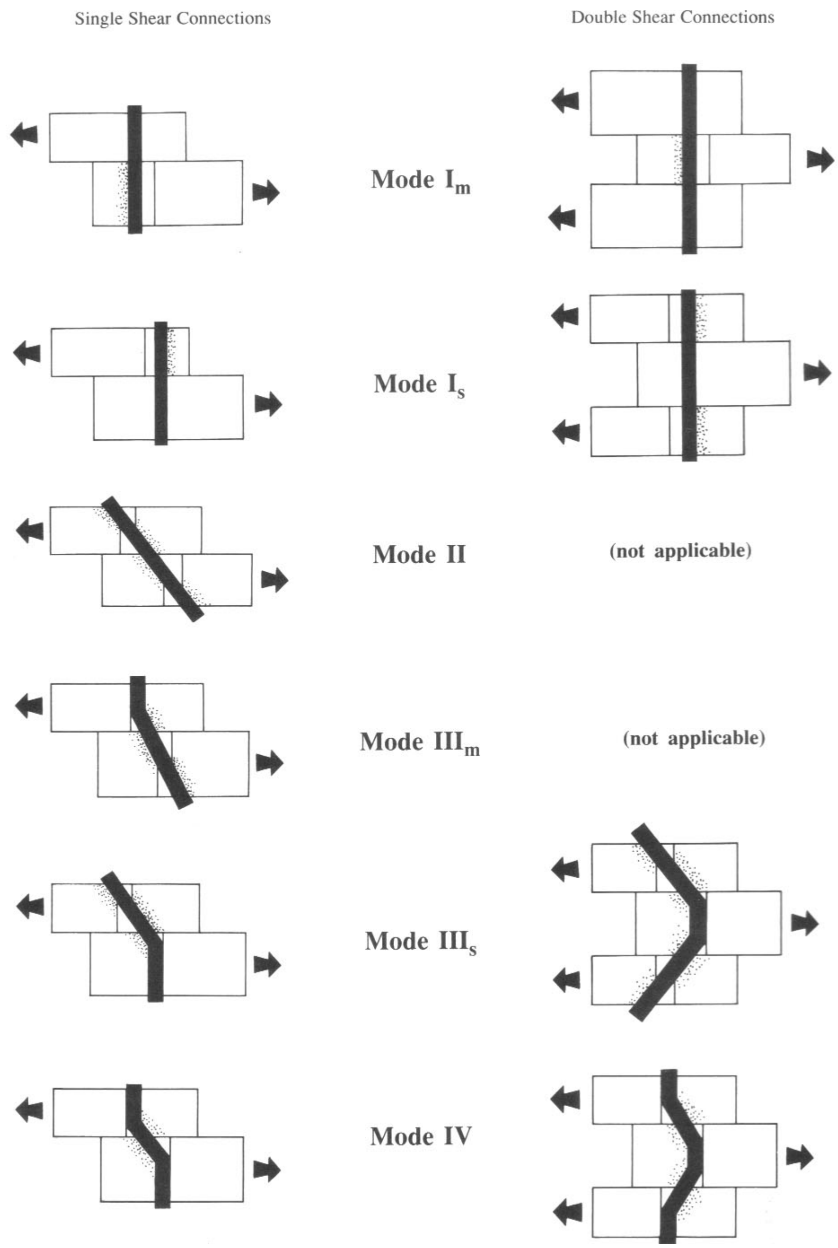


Figure 5.4 - Yield modes for single and double shear timber connections with metal dowels from National Design Specification for Wood Construction (AF&PA 2005)

Since mortise and tenon connections are always double shear connections, this is the focus of the majority of the research performed on timber dowel connections. A discussion of the applicability to Town lattice truss connections, none of which are pure double-shear connections, will follow.

Two proposed sets of failure mechanisms for double shear connections with timber dowels are shown in Figure 5.5 and Figure 5.6.

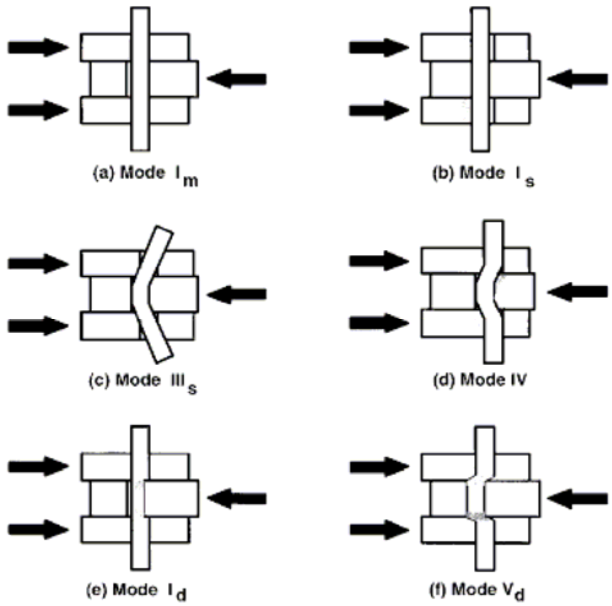


Figure 5.5 - Wooden peg failure mechanisms from Sandberg et al. (2000)

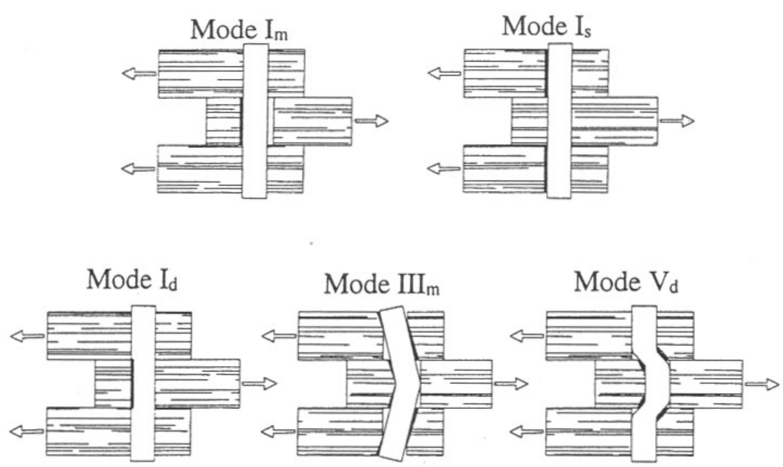


Figure 5.6 - Wooden peg failure mechanisms from Schmidt and Daniels (1999)

Both sets of proposed yield modes exclude the single shear only modes (**II** and $\mathbf{III_m}$) from NDS and include two new dowel failure modes ($\mathbf{I_d}$ and $\mathbf{V_d}$). NDS only includes dowel failure in bending as is consistent with metal dowels. Mode $\mathbf{I_d}$ and $\mathbf{V_d}$ represent bearing failure of the dowel and shearing failure of the dowel, respectively, which are two modes that are unique for wooden dowels.

The main difference between the two sets of proposed modes is in the method of dealing with bending failure of the wooden peg itself. Sandberg includes the NDS mechanisms ($\mathbf{III_s}$ and **IV**) and uses their associated formulae while Schmidt and Daniels include only a single hinge bending mechanism (**III$_m$** in Figure 5.6 though originally more aptly dubbed **III$_s$'** by Schmidt and Mackay (1997))

5.1.1.2 - Yield Mode Equations

The equations to predict the yield loads of a single-peg joint for the mechanisms shown above will now be presented. All equations calculate the average yield load, which can be used for prediction purposes but must be scaled down by a safety factor to be used in design.

In general, equations will consist of terms for both geometric parameters and material yield strength properties. As wood does not typically exhibit bi-linear mechanical behaviour, the determination of yield load must be prescribed. The convention in research focusing on the determination of yield strength of wood for use in doweled connections is to extract yield load from the load-deformation curve using the 5% offset method, as described in ASTM D5652 – Standard Test Methods for Bolted Connections in Wood and Wood-Based Products (ASTM 2000). The 5% offset method defines the yield load as the intercept of the load-deformation curve and a line parallel to the initial elastic slope and offset by 5% of the bolt diameter, as shown in Figure 5.7.

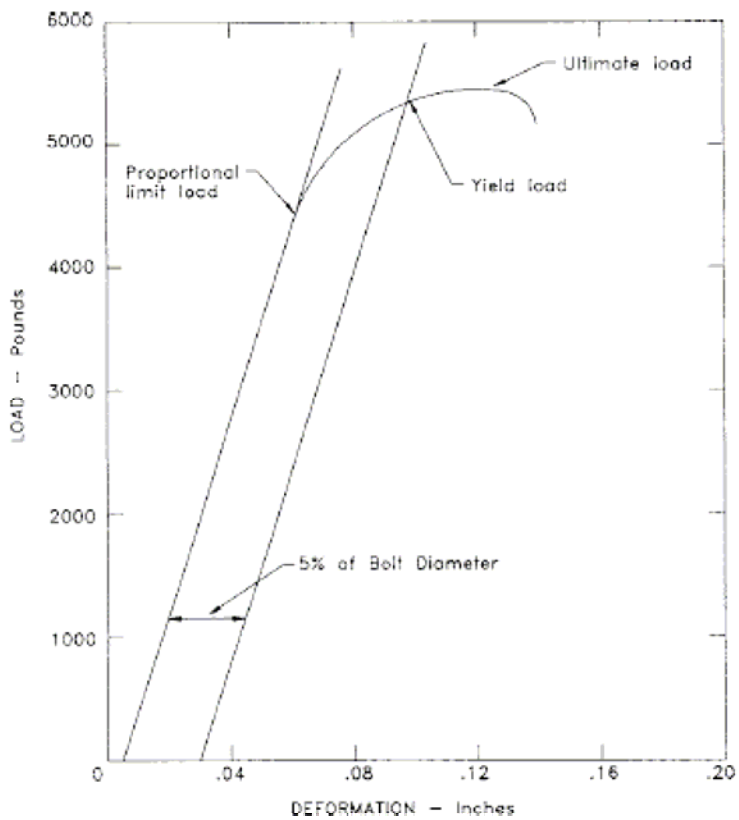


Figure 5.7 - Sample load-deformation curve illustrating 5% offset method for yield load determination (ASTM 2000)

It has been shown that many mechanical properties of wood, including strength, can be related back to simple material properties, such as specific gravity, and work has been done by a variety of researchers to relate the strength properties above to simple geometric and material properties. These results are empirical and more work should be done to gain a higher confidence in the relationships, especially for those properties that are unique to wooden pegged connections. For the present, however, they offer the best option for the prediction of connection strength.

Specific gravity for wood should always be accompanied by an associated moisture content. The measured density of wood will change as the moisture content of the wood changes, affecting the meaning of any measurement result. Specific gravities at different moisture contents can be converted and compared using a standard chart provided in ASTM D2395 - Standard Test Methods for Specific Gravity of Wood and Wood-Based Materials (ASTM 2007).

When relating mechanical properties to specific gravity, it is common practice to use either specific gravity at 12% moisture content, G_{12}, or dry specific gravity (at 0% moisture content), G_d. In the equations below, all specific gravities have been converted to dry values, if necessary, based on the following relationship between dry specific gravity and specific gravity at 12% moisture content proposed by Wilkinson (1991)

$$G_d \cong 1.067 \cdot G_{12}$$

5.1.1.2.1 - Bearing Modes

Three of the modes in each set represent bearing failures and equation are derived based on a maximum bearing stress applied over a projected area.

Mode $\mathbf{I_m}$ $\quad Z = D \cdot t_m \cdot F_{em}$

Mode $\mathbf{I_s}$ $\quad Z = 2 \cdot D \cdot t_s \cdot F_{es}$

Mode $\mathbf{I_d}$ $\quad$ lesser of $Z = D \cdot t_m \cdot F_{ed}$ or $Z = 2 \cdot D \cdot t_s \cdot F_{ed}$

where t_m and t_s are the thicknesses (in) of the main (thicker or centre) member and the side (thinner or outside) member or members, respectively, D is the dowel diameter (in), F_{em} and F_{es} are the dowel bearing strengths (psi) of the main member and side member or members, respectively, and F_{ed} is the bearing strength (psi) of the dowel.

In the formulation above, it is assumed that the bearing strength of each material can be considered separately. Dowel bearing strength values (F_{em} and F_{es}) are defined in NDS and based on loading a member material in bearing with an incompressible dowel set in an appropriately sized drilled hole, as illustrated in Figure 5.8. Bearing strength of the dowel (F_{ed}) was proposed by Schmidt and Daniels (1999) and measured by compressing wooden dowels between incompressible blocks with appropriately rounded seats, as illustrated in Figure 5.9.

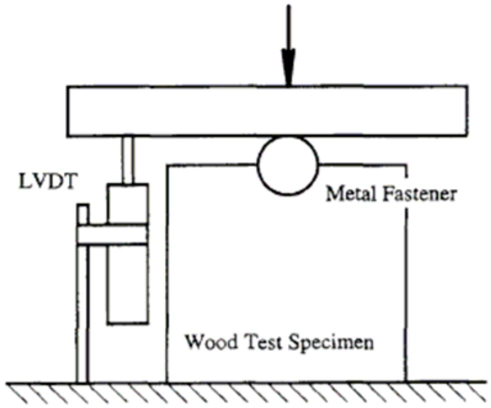


Figure 5.8 - Test apparatus for determination of dowel bearing strength (Church and Tew 1997)

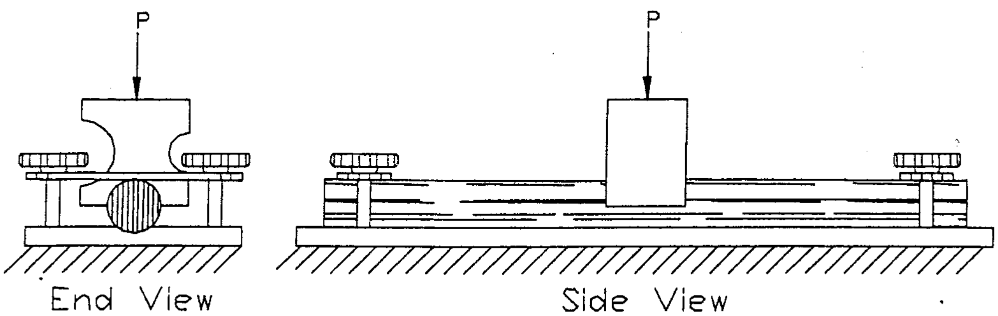


Figure 5.9 - Test apparatus for determination of bearing strength of wooden dowels (Schmidt and Daniels 1999)

An interpretation of the results from Church and Tew (1997) and Schmidt and Daniels (1999) suggests that bearing strengths determined using an incompressible loading apparatus on the base material and dowel separately can be used to provide a reasonable estimate of the combined bearing strength. There is evidence that this is valid if the yield loads for the components are significantly disparate, though there is some question if the results can be extended to assemblies of components with similar yield loads. For this work, it is assumed that yield loads will be significantly different since the grain-orientations of the dowels and members will be different relative to the loading direction. Members will be consistently loaded parallel to grain, an orientation that typically shows a higher bearing strength, while dowels will be consistently loaded perpendicular to grain, an orientation that typically shows a lower bearing strength.

For general design application, equations are needed relating base material dowel bearing strength to base material specific gravity and relating peg bearing strength to peg specific gravity. The NDS proposes equations for the dowel-bearing strength of the wooden members based on the dry specific gravity of the member material. For this work, only the dowel-bearing strength loaded parallel to grain will be used.

$F_{em} = 11200 \cdot G_{dm}$ (Wilkinson)

Schmidt and Daniels have conducted the only known research on bearing strength of the pegs loaded with an incompressible loading apparatus.

$F_{ed} = 5300 \cdot G_{dd}^{2.04}$ (Schmidt and Daniels)

In the Standard for Design of Timber Frame Structures (TFEC 2007), only $\mathbf{I_m}$ and $\mathbf{I_s}$ are included, but F_{em} and F_{es} are required to be based on "the combined response of the timber and wood peg fasteners used in the connection." The required testing this implies may be feasible for an individual structure where the materials are known, but is problematic for use in general design. Few results exist for the combined response of wooden pegs in bearing and no resulting predictive equations have been developed. This lack of data was the motivation behind the decision by Schmidt and Daniels to treat the components individually. For the same reasons, this decision will be also followed herein.

5.1.1.2.2 - Bending Modes

Both of the proposed sets of yield modes include failure of the peg in bending. Sandberg et al. follow NDS, while Schmidt and Daniels propose a new single hinge yield mode. As Schmidt and Daniels never develop an equation to define the new mode, the NDS equations will be used.

Mode $\mathbf{III_s}$
$$Z = \frac{2 \cdot D \cdot t_s \cdot F_{em} \cdot F_{es}}{2 \cdot F_{es} + F_{em}} \cdot \left(\sqrt{Q} - 1\right)$$

where
$$Q = \frac{2 \cdot \left(F_{es} + F_{em}\right)}{F_{em}} + \frac{2 \cdot F_{yb} \cdot \left(F_{em} + 2 \cdot F_{es}\right) \cdot D^2}{3 \cdot F_{em} \cdot F_{es} \cdot t_s^2}$$

Mode $\mathbf{IV}$
$$Z = 2 \cdot D^2 \sqrt{\frac{2 \cdot F_{em} \cdot F_{es} \cdot F_{yb}}{3 \cdot \left(F_{es} + F_{em}\right)}}$$

where F_{yb} is bending yield strength (psi) of the dowel. Flexural properties for timber members are typically determined following procedures defined in ASTM D198 - Standard Test Methods of Static Tests of Lumber in Structural Sizes. A standard apparatus used for flexure testing and the determination of bending yield strength is shown in Figure 5.10.

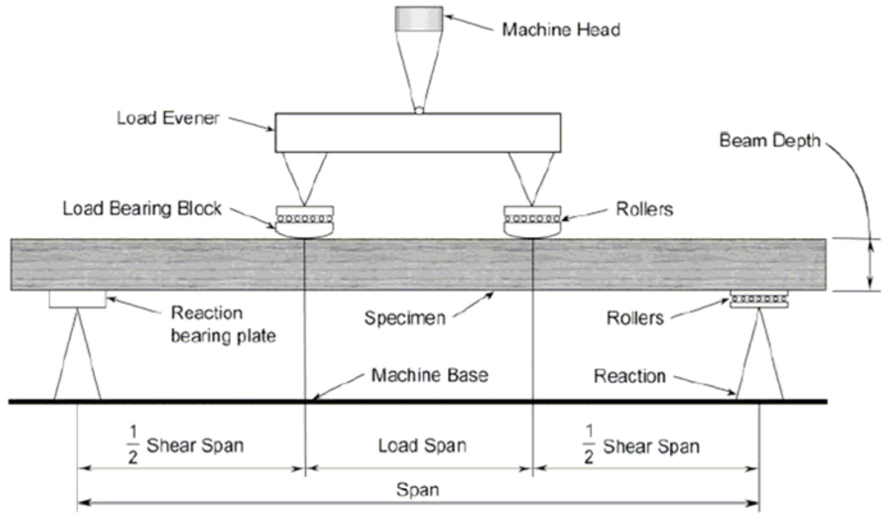


Figure 5.10 - Example of two-point loading apparatus for determination of bending strength (ASTM 2005)

Bending strength values have been found specifically for round wooden pegs (Schmidt and Daniels) and for general wood (Wood Handbook (Forest Products Laboratory 1999))

$F_{yb} = 23830 \cdot G_{dd}^{1.87}$ (Schmidt and Daniels)

$F_{yb} = 23100 \cdot G_{dd}^{1.13}$ (Wood Handbook)

5.1.1.2.3 - Shear Mode

Both sets of proposed modes include a peg shear failure, which is unique for wooden pegs. Wood has low shear strength to bending strength ratio when compared with metal. This is a result of the orthotropic nature of wood and a resulting low shear strength parallel to the grain of the wood. At shear failure, the wooden peg sees a separation of between its fibers, resulting in a disconnected bundle of small tubes as opposed to a single continuous cross-section. This type of failure is not possible with a metal dowel, and subsequently not accounted for in the NDS.

The standard wooden peg connection shear strength equation is based on an average shear strength over of the area of the dowel.

Mode $\mathbf{V_d}$ $$Z = 2 \cdot \frac{\pi \cdot D^2}{4} \cdot F_{ev}$$

where F_{ev} is cross-grain shear strength (psi) of dowel.

Schmidt and Mackay and Schmidt and Daniels both found values for dowel shear strength experimentally. The loading apparatus used for the tests is shown in Figure 5.11. Pegs were held in incompressible blocks with an adjustable shear span, *a*. This spacing was intended to model the varying lengths of shear failure seen in tested connections.

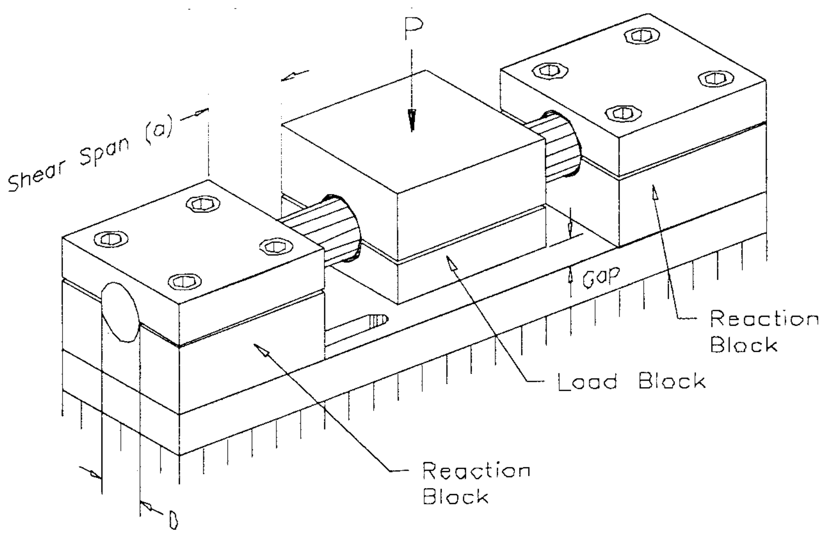


Figure 5.11 - Wooden dowel shear strength loading apparatus (Schmidt and Mackay 1997)

Based on the experimental results, Schmidt and Daniels developed empirical relationships between cross-grain shear strength and dowel specific gravity for a variety of shear span to peg diameter ratios.

$$F_{ev} = 3165 \cdot G_{dd}^{1.06} \text{ for } a = 1/8 \cdot D \quad \text{(Schmidt and Daniels)}$$
$$F_{ev} = 2682 \cdot G_{dd}^{0.88} \text{ for } a = 1/4 \cdot D \quad \text{(Schmidt and Daniels)}$$
$$F_{ev} = 2415 \cdot G_{dd}^{0.84} \text{ for } a = 1/2 \cdot D \quad \text{(Schmidt and Daniels)}$$
$$F_{ev} = 1830 \cdot G_{dd}^{0.56} \text{ for } a = 1 \cdot D \quad \text{(Schmidt and Daniels)}$$

To apply these equations, one would need to determine the appropriate shear span for any given combination of member and peg wood. General relationships for this have not been developed.

Miller and Schmidt attempted to solve this lack of knowledge of appropriate shear span by developing a direct relationship between member and dowel specific gravity and cross-grain shear strength. This empirical relationship was based on a combination of experimental results and results from a finite element model of a wooden peg connection.

$$F_{ev} = 4810 \cdot G_{dd}^{0.926} \cdot G_{dm}^{0.778} \quad \text{(Miller and Schmidt average)}$$

The major limitation of this equation is the lack of a term for peg diameter. Because of this, the relationship is only considered valid for peg diameters between 0.75 and 1.25 inches. Since pegs used in Town lattice truss bridges generally have diameters larger than this range, there is a need for further analysis.

Equating the equation from Miller and Schmidt with those from Schmidt and Daniels allows for the development of a relationship between dowel and member specific gravity for each shear span. These results are plotted in Figure 5.12.

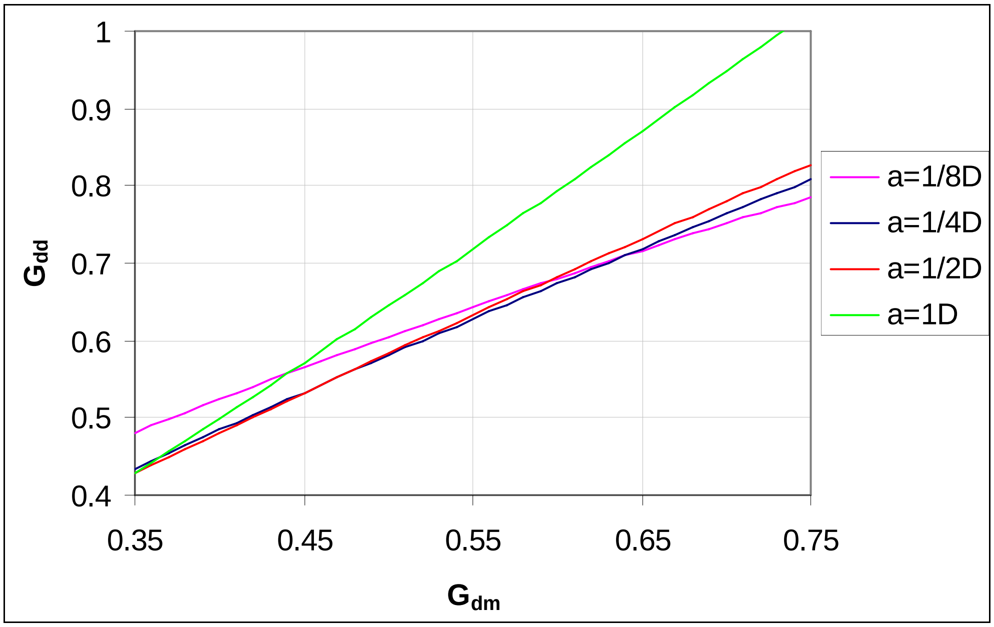


Figure 5.12 - Empirical relationship between dowel and member specific gravity for various shear spans

Unfortunately, the resulting relationships shown in Figure 5.12 do not offer any clear direction on how to select the appropriate shear span. This means the equation from Miller and Schmidt cannot be applied for this work and the Schmidt and Daniels equations based on shear span will be considered as the best current option for the prediction of shear strength for larger diameter pegged connections.

5.1.2 - Verification of equations from literature

Since a limited number of tests have been performed for a variety of the formulae proposed in the previous section, it is considered worthwhile to verify the applicability through comparison with experimental results.

Sandberg et al. (2000) performed tests on a variety of simulated mortise and tenon connections and compared the strength and yield mode with the predicted strength and yield modes suggested through the formulae presented above. One exception is in the calculation of peg shear strength for the $\mathbf{V_d}$ mode, where the author extrapolated a peg shear strength value for zero span from Schmidt and Daniels' shear strength results. The results of the work are shown in Table 5.1.

Table 5.1 – Comparison of experimental strengths to modified yield model from Sandberg et al. (2000)

	Experimental		Modified European Yield Model Predictions (kN)					
Group (1)	Strength (kN) (2)	Mode (3)	Mode I_m (4)	Mode I_s (5)	Mode I_d (6)	Mode III_s (7)	Mode IV (8)	Mode V_d (9)
PT00M1	15.7	I_s	38.9	13.9	17.7	22.0	30.3	16.2
PT90M1	15.3	I_m	13.9	38.9	17.7	22.3	30.3	16.2
PT00M2	15.9	III_s/V_d	38.9	27.7	17.7	21.8	30.3	16.2
PT90M2	16.5	I_m	13.9	77.7	17.7	28.0	30.3	16.2
MT00M1	17.9	I_d/V_d	69.7	32.4	17.7	31.6	44.6	16.2
MT90M1	18.7	I_d/V_d	32.4	69.7	17.7	34.5	44.6	16.2
MT00M2	18.3	I_d/V_d	69.7	64.7	17.7	35.8	44.6	16.2
MT90M2	17.1	I_d/V_d	32.4	139.5	17.7	48.1	44.6	16.2

All specimens had two 1” diameter red oak pegs through a 2” wide tenon (shown in Figure 5.16). Members were fabricated either from eastern white pine or sugar maple, species that were selected to frame the range of likely specific gravities used in traditional timber construction. Wood species is indicated in the group identification as either ‘P’ for eastern white pine or ‘M’ for sugar maple. Specimens were constructed with mortise grain and tenon grain orthogonal to each other and with the direction of the tenon grain either parallel or perpendicular to the loading as indicated by ‘00’ or ‘90’ in the group identification. Finally, mortise members were either 1” or 2” wide as indicated by ‘M1’ or M2’ in the group identification.

The results shown in Table 5.1 support the validity of the modified yield model. The correct failure mechanism is correctly indicated by the lowest yield force in all cases and the yield forces offer reasonably close predictions of experimental yield strength. In addition, the results underline the importance of the failure mechanisms unique to wooden pegged connections, $\mathbf{I_d}$ and $\mathbf{V_d}$, which dominate in all cases except those where a weak member wood is loaded in bearing perpendicular to grain (PT00M1, PT90M1, and PT90M2). Even in these cases, it can be seen that the yield loads for modes $\mathbf{I_d}$ and $\mathbf{V_d}$ are not significantly higher, and it is expected that a small increase in member specific gravity or a small decrease in peg specific gravity is likely to make one of those modes dominate. Thus, it is considered very important to have reliable prediction rules for these modes. Unfortunately, these are the modes that have the least associated research.

To further a assess the applicability of the equations presented above, the predicted yield loads were calculated for models used in several experimental studies of wooden pegged connections (Kessel and Augustin 1994; McFarland-Johnson 1995; Burnett et al. 2003) and compared with the experimental results from these studies. The results of this work can be seen in Table 5.2. Details on the experimental tests and calculations can be found in Appendix C.

Table 5.2 - Comparison of experimental strengths to modified yield model predictions (experimental results from McFarland-Johnson (1995), Burnett et al. (2003), and Kessel and Augustin (1994))

Experimental		Predicted								
Strength (lbs)	Mode	Mode $\mathbf{I_m}$ (lbs)	Mode $\mathbf{I_s}$ (lbs)	Mode $\mathbf{I_d}$ (lbs)	Mode $\mathbf{III_s}$ (lbs)	Mode **IV** (lbs)	Mode $\mathbf{V_d}$ (lbs) a=1D	a=1/2D	a=1/4D	a=1/8D
McFarland-Johnson: 1 Peg Parallel										
10800	$\mathbf{V_d}$	24150	48300	14123	23299	27805	7504	9044	$\boxed{10247}$	$\boxed{11353}$
McFarland-Johnson: 3 Peg Parallel										
26286	$\mathbf{V_d}$	72450	144900	42368	69896	83415	$\boxed{22513}$	$\boxed{27132}$	30740	34059
McFarland-Johnson: 1 Peg Perpendicular										
8500	$\mathbf{V_d}$	24150	13290	14123	12918	18266	$\boxed{7504}$	$\boxed{9044}$	10247	11353
McFarland-Johnson: 3 Peg Perpendicular										
25833	$\mathbf{V_d}$	72450	39871	42368	38755	54799	$\boxed{22513}$	$\boxed{27132}$	30740	34059
Burnett et al.: 1 Peg Perpendicular - Douglas Fir										
3590	unknown	9408	8272	5299	5916	8133	2465	3012	$\boxed{3309}$	$\boxed{3717}$
Burnett et al.: 1 Peg Perpendicular - Eastern White Pine										
3274	unknown	7448	5249	5299	4737	6685	2465	$\boxed{3012}$	$\boxed{3309}$	3717
Burnett et al.: 1 Peg Perpendicular - Red Oak										
3607	unknown	14112	13260	5299	7863	10191	2465	3012	$\boxed{3309}$	$\boxed{3717}$
Kessel and Augustin: 2 Peg Perpendicular - 24mm Oak										
4518	$\mathbf{V_d}$[1]	24998	20034	8977	11901	15411	$\boxed{4136}$	$\boxed{4899}$	5358	5898
Kessel and Augustin: 2 Peg Perpendicular - 32mm Oak										
8228	$\mathbf{V_d}$[1]	53330	30845	19151	19610	26039	$\boxed{7353}$	$\boxed{8710}$	9525	10486
Kessel and Augustin: 2 Peg Perpendicular - 40mm Oak										
11735	$\mathbf{V_d}$[1]	66663	34486	23939	27907	39057	$\boxed{11488}$	$\boxed{13609}$	14882	16385
Kessel and Augustin: 2 Peg Perpendicular - 24mm Spruce										
3912	$\mathbf{V_d}$[1]	16666	11129	8977	8490	11793	4136	4899	5358	5898
Kessel and Augustin: 2 Peg Perpendicular - 32mm Spruce										
6070	$\mathbf{V_d}$[1]	35553	17134	19151	14171	19874	7353	8710	9525	10486

[1] peg failures were Mode $\mathbf{V_d}$, but mortise and relish failures also occurred at similar load levels

As can be seen, the equations again yield reasonable predictions of yield load and suggest that mode $\mathbf{V_d}$ is the likely dominant failure mechanism. All experimental results in which failure mode was recorded demontrated shear failures and the yield mode equations consistently predict $\mathbf{V_d}$ to have the lowest yield load. A lack of information on the appropriate shear span makes an exact comparison difficult and underlines the need for more work in developing and assessing the prediction model for shear failure.

From the comparison above, it seems that the modified yield model equations as presented above offer reasonable predictions for failure mechanism and yield load for the research being performed.

5.1.3 - Town Lattice Truss Connections

The material strength equations can be combined into the yield load equations to create a set of equations based only on geometric properties and the specific gravities of the woods. The set of equations can be further simplified by making some basic assumptions based on the standard layout of the Town lattice truss connection. Side and main members can be taken to have equal thickness, equal specific gravity, and orientation with grain parallel to loading. This will yield a set of equations based on only four variables, G_{dm}, G_{dd}, t, and D.

For $\mathbf{V_d}$, a decision on shear span must be made. Based on the results in Table 5.2, and in particular those from McFarland-Johnson who conducted the only known testing of pegged connections for the Town lattice truss bridge, a = 1/2D is seen to yield the best prediction on average and will be used throughout the remainder of this work.

Mode $\mathbf{I_m}$ $\quad ZI_m = D \cdot t \cdot 11200 \cdot G_{dm}$

Mode $\mathbf{I_d}$ $\quad ZI_d = D \cdot t \cdot 5300 \cdot G_{dd}^{2.04}$

Mode $\mathbf{III_s}$ $\quad ZIII = 7467 \cdot D \cdot t \cdot G_{dm} \cdot \left(\sqrt{Q} - 1\right)$ where $Q = 4 + \dfrac{47660 G_{dd}^{1.87} \cdot D^2}{11200 G_{dm} \cdot t^2}$

Mode $\mathbf{IV}$ $\quad ZIV = 18864 \cdot D^2 \cdot G_{dm}^{0.5} \cdot G_{dd}^{0.935}$

Mode $\mathbf{V_d}$ $\quad ZV_d = \dfrac{\pi \cdot D^2}{2} \cdot 2415 \cdot G_{dd}^{0.84}$ for $a = 1/2 \cdot D$

Ratios of the above equations can be taken, creating a comparison factor. Foe example, comparison factor 1, CF_1, relates mode $\mathbf{I_m}$ and mode $\mathbf{I_d}$, yielding the equation

$$CF_1 = \frac{ZI_m}{ZI_d} = \frac{D \cdot t \cdot 11200 \cdot G_{dm}}{D \cdot t \cdot 5300 \cdot G_{dd}^{2.04}} = 2.113 \cdot \frac{G_{dm}}{G_{dd}^{2.04}}$$

If CF_1 is greater than 1, then ZI_m is greater than ZI_d, meaning ZI_d dominates the yielding of the connection. If CF_1 is less than 1, then ZI_m is less than ZI_d, meaning ZI_m dominates the yielding of the connection. Setting CF_1 equal to 1 yields a relationship between G_{dm} and G_{dd} that defines all situations where mode $\mathbf{I_m}$ and mode $\mathbf{I_d}$ have the same yield load.

$$CF_1 = 1 = 2.113 \cdot \frac{G_{dm}}{G_{dd}^{2.04}}; \quad G_{dm} = 0.473 \cdot G_{dd}^{2.04} \text{ or } G_{dd} = 1.443 \cdot G_{dm}^{0.49}$$

This type of comparison factor can be created for all permutations of the yield modes presented above. Some of the relationships can be solved explicitly, as was true for CF1, while others must be solved implicitly. The results can be plotted, as shown in Figure 5.13 for G_{dm} = 0.3. Each curve represents a pair of modes whose yield loads are equal. Thus, the area above or below a curve represents the domination of one of the modes over the other. In Figure 5.13, the area above the curve represents the domination of the first mode listed in the equality and the area below the curve represents the domination of the second mode listed in the equality. These relative dominations can be combined logically to find the yield mode that dominates absolutely in any particular area. This result is shown in Figure 5.14 by superimposing shaded areas on the original graph. Calculations for comparison factors and equalities are included in Appendix D.

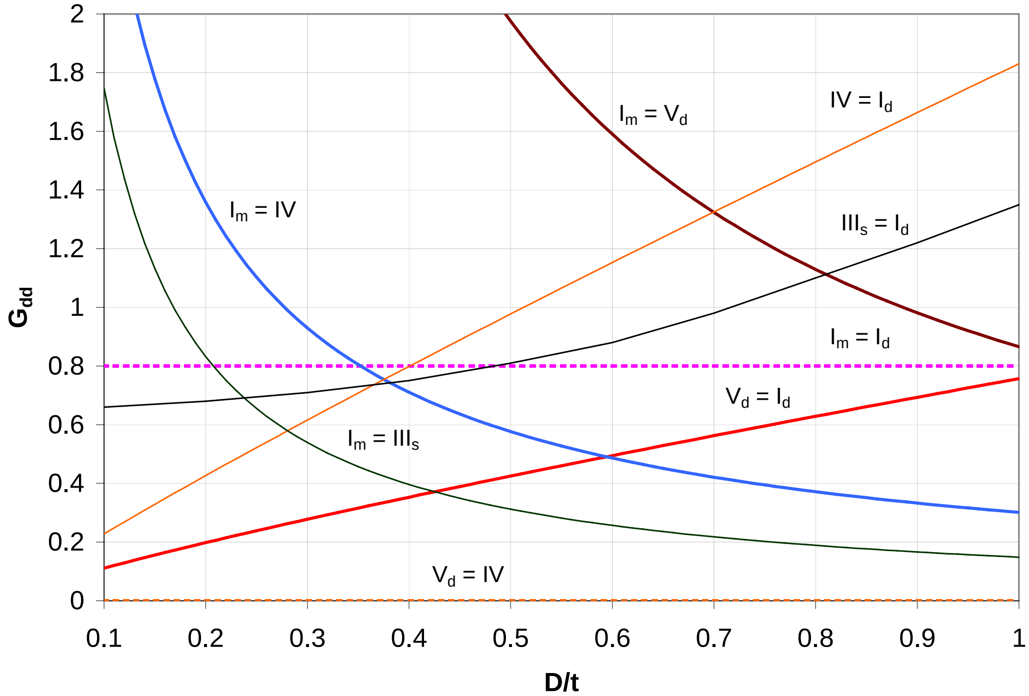


Figure 5.13 – Graph of failure mechanism equalities with $G_{dm} = 0.3$

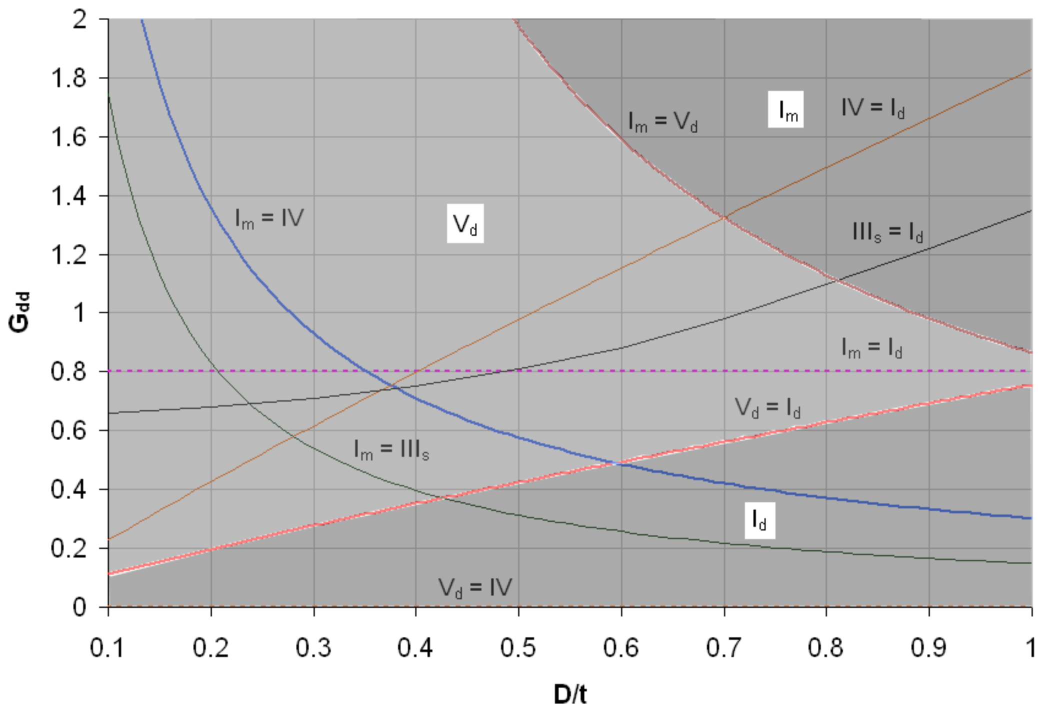


Figure 5.14 - Dominating yield modes $G_{dm} = 0.3$

It should be noted that Figure 5.13 and Figure 5.14 are largely for illustration purposes and include a range of G_{dd} values beyond what is reasonable for the specific gravity of wood. This was done to include more of the curves of equality. Despite this, the equality curve for Mode **III$_s$** and Mode **V$_d$** is still outside the range of the plot. The range of *D/t* values was selected to encompass a typical range used in traditional timber frame construction and the Town lattice truss. Figure 5.15 shows the resulting yield mode plots for different shear spans when a more realistic range of G_{dd} values is used.

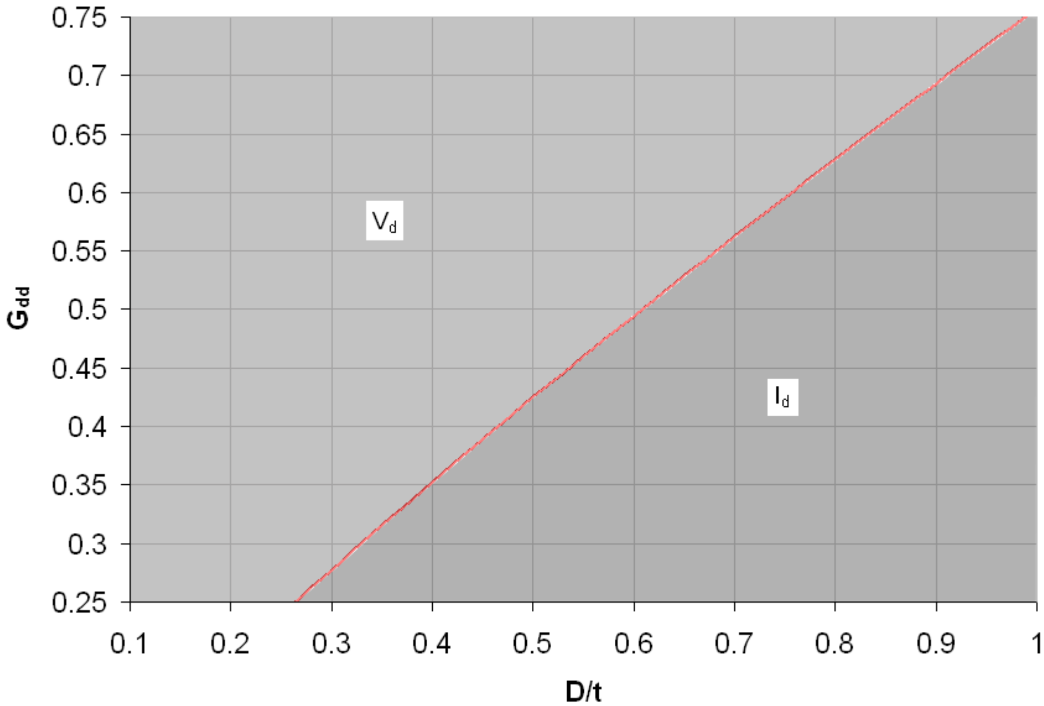


Figure 5.15 - Dominating yield modes for Town lattice truss connections

The plot in Figure 5.15 shows only two failure modes, Mode **I$_d$** and Mode **V$_d$,** which are the two modes specific to wooden dowelled connections and specifically relate to a failure in the peg as opposed to the member. Note that this causes G_{dm} to not factor into the results. Based on the plots, it is expected that the connections in the Town lattice truss will be typically dominated by these two failure modes.

5.1.4 - Strength results

Based on the results above, the strength of wooden pegged connections as generally used in the Town lattice truss will be dominated by two wooden dowel specific failure modes, bearing failure in the dowel (Mode **I$_d$**) and shear failure in the dowel (Mode **V$_d$**). Combining theoretical yield load equations with empirical equations for material strength gives design yield load equations based on geometric properties and the specific gravity of the peg only. Alternately, material strength properties can be used directly is they are available from material testing.

Mode $\mathbf{I_d}$ $\quad Z = \dfrac{5300 \cdot D \cdot t \cdot G_{dd}^{2.04}}{C_d \cdot R_d}$ or $Z = \dfrac{D \cdot t \cdot F_{ed}}{C_d \cdot R_d}$

Mode $\mathbf{V_d}$ $\quad Z = \dfrac{3793 \cdot D^2 \cdot G_{dd}^{0.84}}{C_d \cdot R_d}$ or $Z = \dfrac{2 \cdot \pi \cdot D^2 \cdot F_{ev}}{4 \cdot C_d \cdot R_d}$

C_d is a load duration factor that accounts for the short time duration of the tests used to generate material strength properties and R_d is a reduction factor that accounts for reducing the design load below a certain percent exclusion value and a safety factor. A load duration factor value of 1.6 should be used and is based on a 10-minute load duration, as given in Table 3.4. The NDS proposes a reduction factor value of 2.5 for bearing and Miller and Schmidt propose a value of 2.2 for shear.

Finally, the strength results yielded above for double shear connections must be converted into values that can be used in the Town lattice truss. The chord connections in the Town lattice truss have wooden pegs that connect six members, which can all be loaded in different directions and with different magnitudes. Thus, these connections are neither simple double shear joints since they are not loaded symmetrically, neither are they single shear joints since there are generally more than two members offering support to the peg. Without any experimental results to verify the actual behaviour of these connections, it is assumed that each shear plane between members along the length of the peg can be modeled independently and will behave as one half of a double shear connection, which is consistent with the work performed by McFarland-Johnson (1995). Therefore, the strength of one such connection will be taken to be one half of the value of the equivalent double shear joint. This yields final design strength equations for single pegs in Town lattice truss connections.

Mode $\mathbf{I_d}$ $\quad ZI_d = 662.5 \cdot D \cdot t \cdot G_{dd}^{2.04}$ or $ZI_d = \dfrac{D \cdot t \cdot F_{ed}}{8}$

Mode $\mathbf{V_d}$ $\quad ZV_d = 538.8 \cdot D^2 \cdot G_{dd}^{0.84}$ or $ZV_d = 0.223 \cdot D^2 \cdot F_{ev}$

5.2 - Connection Stiffness

The objective of this stiffness analysis is to develop an understanding of the behaviour of wooden pegged connections under low-level loads. The most important outcome of this analysis will be an estimation procedure to generate stiffness properties for the joints in a Town lattice truss, to be used in further analysis of larger components of the structure.

With the inherent variability in the properties of wood, the likely inconsistency between construction details and execution, and the dearth of previous research on wooden pegged connections, exact stiffness values will be impossible to obtain, and would likely not be accurate for a given structure. A rough estimate of the values is the most that can be expected but will still be a contribution since no generally applicable prediction procedures exist.

Literature on stiffness in wooden pegged connections is generally the same as literature for strength, although more limited since much of the research focuses exclusively on strength and does not provide any stiffness values. McFarland-Johnson (1995) offer some stiffness-related results specifically for Town lattice truss connections and a number of researchers offer stiffness-related results for mortise and tenon connections in traditional timber construction (Schmidt and Mackay 1997; Schmidt and Daniels 1999; Sandberg et al. 2000; Burnett et al. 2003). The double-shear nature of the mortise and tenon connections and the fact that peg diameters are typically smaller than those used in the Town lattice truss make this secondary research not directly applicable. However, the results will be used in developing an analytical model that can be used to predict low-level load behaviour of the connections in the Town lattice truss.

5.2.1 - Literature

The goal of most testing of wooden pegged connections is to determine strength parameters. Stiffness is generally a secondary concern, and only some of the literature on wooden pegged connections includes stiffness data. Three experimental programs that present stiffness results will be addressed herein as comparisons with a developed analytical predictive model.

Sandberg et al. (2000) tested wooden pegged connections as part of a study of the behaviour of mortise and tenon connections for traditional timber framing. Eight sets of specimens were tested with 1" diameter red oak pegs and varying member wood (pine or maple), member grain orientation (tenon parallel and mortise perpendicular or vice versa), and relative mortise-to-tenon thickness (2" tenon with 1" or 2" mortise). Strength results were compared with the modified yield model, as presented in Table 5.1, and stiffness results were compared with an analytical model, presented below. Dimensions for the specimens are shown in Figure 5.16.

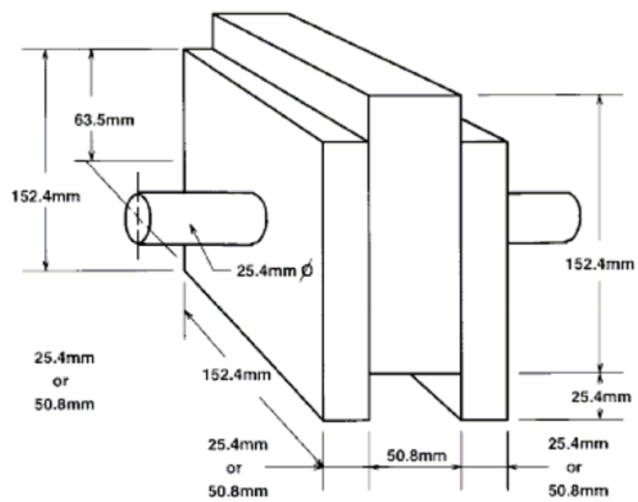


Figure 5.16 – Specimen dimensions (reproduced from Sandberg et al. (2000))

While investigating the effect of end distance on the strength of pegged timber connections, Burnett et al. (2003) found peg stiffnesses for perpendicular members of either eastern white pine or Douglas fir, attached with a single 1" diameter northern red oak peg loaded in double shear. Mean values of stiffness were obtained for 5 different end distances for both types of wood. No significant difference in the means was found for the different distances, so for this work a rounded arithmetic mean of the means will be taken as a suggested stiffness. Dimensions for the specimens are shown in Figure 5.17.

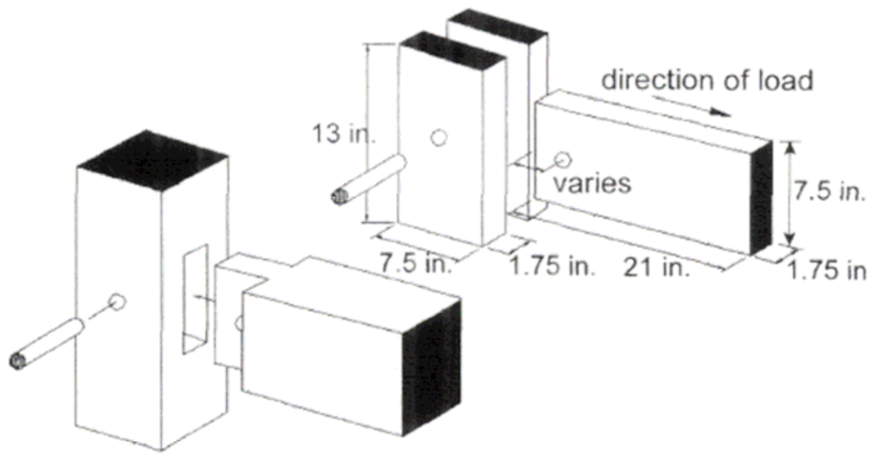


Figure 5.17 – Model 2 specimen dimensions (reproduced from Burnett et al. (2003))

Finally, McFarland-Johnson tested sets of specimens specifically for determination of properties to use in the analysis of Town lattice truss bridges. While most tests were performed with three pegs, several were performed with a single peg centered on the specimens. Specimens were constructed with 1.75″ diameter White Oak pegs connecting two 3″ wide mortise pieces around a 3″ wide tenon piece. Mortise and tenon pieces were made of Eastern White Spruce. A sketch of a general parallel to grain test specimen from the McFarland-Johnson report is shown in Figure 5.18.

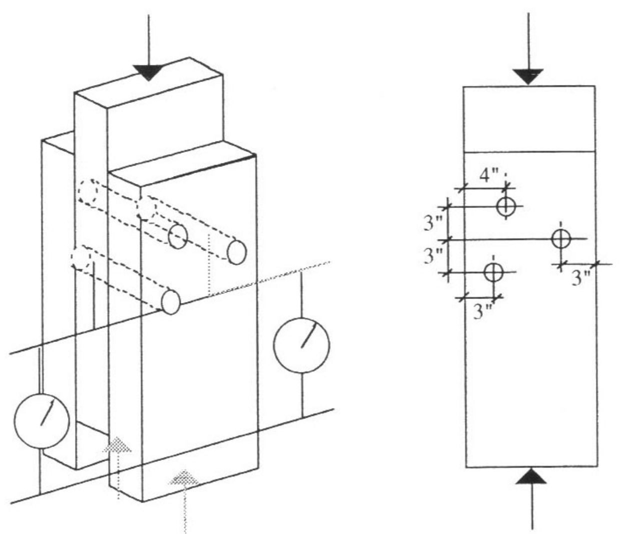


Figure 5.18 - Test setup from McFarland-Johnson (1995)

Only one effort to develop an analytical model for the stiffness of wooden peg joints was found in the literature. Sandberg et al. (2000) proposed a simply-supported beam model to estimate the stiffness of mortise and tenon joints for use in structural analysis of traditional timber frames. The double shear model used is shown in Figure 5.19.

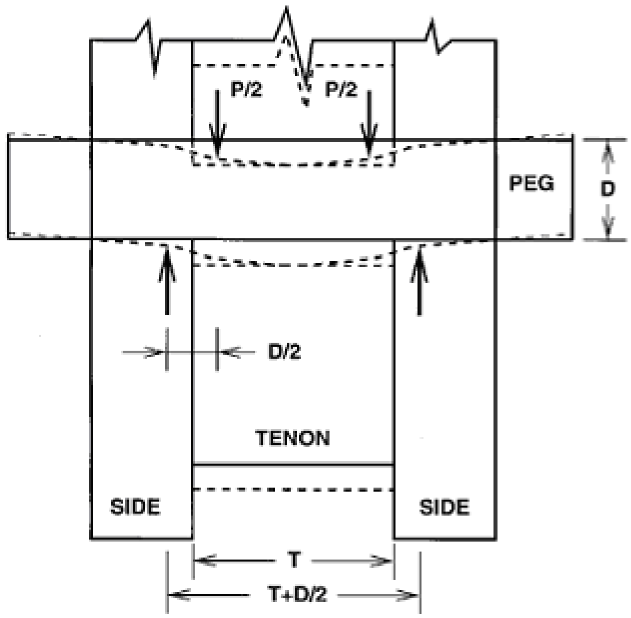


Figure 5.19 - Stiffness model reproduced from Sandberg et al. (2000)

The peg is treated as a symmetric simply supported beam. The applied load from the tenon is represented by two point loads of magnitude $P/2$. The support supplied by the mortises is represented by two point loads equal in magnitude and opposite in direction to the applied loads. The applied and support loads are separated by a distance $D/2$, where D represents the diameter of the peg, centered on the joint of the main and side members. Taking the deflection of the joint to be represented by the displacement of the applied loads leads to a flexibility for the peg of:

$$f_{peg} = \frac{2 \cdot (6 \cdot t - D)}{3 \cdot \pi \cdot E \cdot D^2} + \frac{160}{9 \cdot \pi \cdot E \cdot D}$$

The first term in the result above is the flexibility from bending deformation and the second term is the flexibility from shear deformation. The formulation above assumes a shear modulus equal to $E/16$, which is a standard ratio in wood design, typically used in developing equations for lateral buckling and stability.

The spacing of the loads was based on what was deemed reasonable by the researchers based on post-test deformations. However, the formulation only includes the properties of the peg and does not account for the crushing of the members, which would need to occur to create the shape shown in the Figure 5.19. It seems unreasonable that a peg in a hard wood would have the same stiffness as a peg in a soft wood. The material in the members will in reality exert a distributed pressure, the shape of which will depend on the deformed shape of the peg itself.

Sandberg attempted to account for the effect of the member wood properties by including a flexibility term for the members and deriving its contribution based on the experimental results for full joints. This may help to provide more accurate results, but does not accurately reflect the behaviour of the joint. Furthermore, it does not allow for a decrease in peg flexibility, effectively limiting the joint stiffness based on the peg shape defined above.

To eliminate some of these concerns, a beam on elastic foundation model will be used. The model is appropriate for a pegged connection because force can only be exerted on the peg if the material in the members is deformed, as is the case with the springs in an elastic foundation. These forces and deformations must balance with the resulting shape of the peg, which is based on its internal moments.

5.2.2 - Beam on Elastic Foundation

The beam on elastic foundation model is an analytical tool used in the analysis and design of slabs on grade and suspension bridges. In each case, the main structural element, the slab or bridge deck, is assumed to act as a continuous bending beam and the supporting structure, the soil or the bridge cables, are assumed to act as a continuous distributed elastic stiffness. When transverse shear deformation is neglected and k_s, the distributed stiffness of the foundation, and EI, the bending rigidity of the member, are assumed to be constant, a fundamental differential equation can be derived as:

$$\frac{d^4 v}{dx^4} + \frac{k_s}{EI} v = \frac{\overline{b}}{EI}$$

where v is the vertical deformation and b is the applied external loading. This equation yields a solution of the form

$$v = v_{part} + e^{-\lambda x}(C_1 \sin \lambda x + C_2 \sin \lambda x) + e^{\lambda x}(C_3 \sin \lambda x + C_4 \sin \lambda x)$$

where v_{part} is a particular solution based on the applied loading of displacement, C_1, C_2, C_3, and C_4 are unknown coefficients, and λ is a fundamental shape parameter defined as

$$\lambda = \sqrt[4]{\frac{k_s}{4EI}}$$

By defining four boundary conditions (2 force or deformation conditions at each end), one can solve for the unknown coefficients and find the resulting shape of the beam in the model.

5.2.3 - Analytical Model

All available experimental results for stiffness are based on the double-shear connection, as this is the most common joint seen in the pegged connection of traditional timber framing. None of the connections in the Town lattice truss are actually double-shear connections, however, it is generally accepted to assume each shear plane will behave as half of a double shear connection, as was assumed for connection strength. Therefore, an analytical model will be developed for double-shear pegged connections. The validity of the model will be checked using results of experimental testing, and the results will be used to estimate the stiffness of pegs for Town lattice truss chord connections.

The peg is taken as a beam sitting on a distributed elastic foundation, as shown in Figure 5.20. This elastic foundation represents the member material and will exert pressure proportional and opposite to the displacement at any given location.

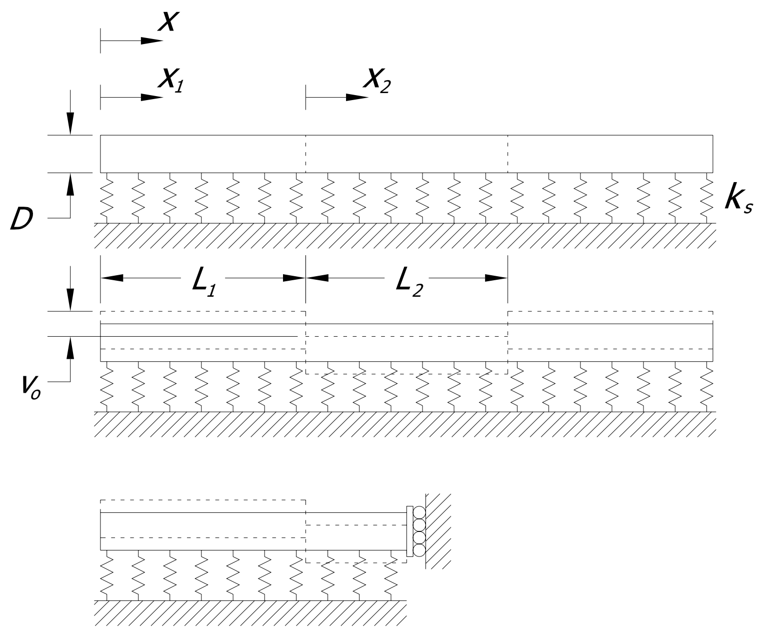


Figure 5.20 - Double shear analytical model

As shown in Figure 5.20, the peg can be divided into three segments based on the member in which each is contained. The outside segments are located within the side members while the inside segment is located within the main member. Each segment will have a defining shape function and local location parameter which must be considered piece-wise and combined together with appropriate continuity and boundary conditions at each member interface.

Since the distribution of applied loading is what is desired, displacements will be imposed on the system. To represent a relative movement of the tenon with respect to the mortise, the equilibrium positions of the springs will be shifted accordingly. The equilibrium position within the mortise is shifted up an amount $v_o/2$ and the equilibrium position within the tenon is shifted down an amount $v_o/2$, resulting in a net displacement between members of v_o, as shown in Figure 5.20.

Since the structure is symmetric, it can be divided along the line of symmetry. In order to do this, an appropriate restraint must be added to take the place of the removed portion of the structure and ensure geometrically consistent behaviour. In the case of a symmetric structure and loading, a fixed roller must be added, which allows translation parallel to the line of symmetry and restricts rotation and translation perpendicular to the line of symmetry. This simplifies the structure to only have two functions, v_1 for the peg within the mortise and v_2 for the peg within the tenon. Taking x_1 and x_2 as the distance from the left end of the mortise and tenon, respectively, and L_1 and L_2 as the width of the mortise and tenon, respectively, one finds the equations below.

$$v_1(x_1) = C_1 \cdot e^{-\lambda x_1} \sin \lambda x_1 + C_2 \cdot e^{-\lambda x_1} \cos \lambda x_1 + C_3 \cdot e^{\lambda x_1} \sin \lambda x_1 + C_4 \cdot e^{\lambda x_1} \cos \lambda x_1 + v_o/2$$
$$v_2(x_2) = C_5 \cdot e^{-\lambda x_2} \sin \lambda x_2 + C_6 \cdot e^{-\lambda x_2} \cos \lambda x_2 + C_7 \cdot e^{\lambda x_2} \sin \lambda x_2 + C_8 \cdot e^{\lambda x_2} \cos \lambda x_2 - v_o/2$$

In order to solve these equations, eight boundary conditions need to be defined. These conditions are given in Table 5.3.

Table 5.3 - Boundary conditions for double shear beam on elastic foundation model

Free End at $x_1 = 0$	
$M_1(0) = 0 \rightarrow \frac{d^2 v_1(0)}{dx_1^{\ 2}} = 0$	$V_1(0) = 0 \rightarrow \frac{d^3 v_1(0)}{dx_1^{\ 3}} = 0$
Fixed Roller at $x_2 = L_2/2$	
$\beta_2(L_2/2) = 0 \rightarrow \frac{dv_2(L_2/2)}{dx_2} = 0$	$V_2(L_2/2) = 0 \rightarrow \frac{d^3 v_2(L_2/2)}{dx_2^{\ 3}} = 0$
Geometric Compatibility at $x_1 = L_1$ and $x_2 = 0$	
$v_1(L_1) = v_2(0)$	$\beta_1(L_1) = \beta_2(0) \rightarrow \frac{dv_1(L_1)}{dx_1} = \frac{dv_2(0)}{dx_2}$
Force Equilibrium at $x_1 = L_1$ and $x_2 = 0$	
$M_1(L_1) = M_2(0) \rightarrow \frac{d^2 v_1(L_1)}{dx_1^{\ 2}} = \frac{d^2 v_2(0)}{dx_2^{\ 2}}$	$V_1(L_1) = V_2(0) \rightarrow \frac{d^3 v_1(L_1)}{dx_1^{\ 3}} = \frac{d^3 v_2(0)}{dx_2^{\ 3}}$

The resulting stiffness of the connection can be calculated as:

$$K = \frac{F}{v_o}$$

where F is the total force exerted on the main member. Since this force must be resisted by the two side member, they must each have a total force exerted on them of $F/2$. Since the only mechanism for force to transfer from the side member to the main member is through shear in the peg, it can then be concluded that the shear at the member interface will be equal to half of the main member force, or:

$$V_1(L_1) = \frac{F}{2} = \frac{K \cdot v_o}{2}$$

If a unit displacement, $v_o = 1$, is applied to the analytical model, then the stiffness of the joint can be extracted from the solution as:

$$K = 2 \cdot V_1(L_1)$$

5.2.4 - Experimental Models

The three experimental models from the literature will be modeled using the beam on elastic foundation analytical model. Numerical values will be needed for each case to find analytical solutions. A number of geometric and material properties are needed for each experimental model. Geometric properties include the widths of the mortise and tenon and the moment of inertia of the peg. Material properties include the modulus of elasticity of the peg and the distributed bearing stiffness of the members.

Geometric properties are straightforward and are given, or can be easily derived, for each of the experimental models. Lengths, L_1 and L_2, are taken to be the thickness of the mortise and tenon, respectively, and moment of inertia, I, can be calculated directly from the peg diameter.

Material properties are somewhat more complicated as they are not directly presented in the works. Modulus of elasticity values for the pegs can be estimated based on values and relationships from the Wood Handbook (Forest Products Laboratory 1999). The relationship for the modulus of elasticity for hardwoods of $E = 2.39 \cdot G_{12}^{0.7} \cdot 10^6$ psi can be used when specific gravity at 12% moisture content, G_{12}, is known. When specific gravity is not given, modulus of elasticity can be taken from tables based on the species of wood.

Finally, a material property is needed for each model to represent the stiffness of the elastic foundation, k_s, which must be linearly distributed. The goal of almost all bearing experiments found in the literature is to find strength parameters. Because of this, only some of the researchers find and present the initial linear force-deformation slope. These overall stiffnesses can be converted into differential stiffnesses by dividing by the effective bearing area of the peg in the test, represented by the product of the diameter of the peg and the width of the testing block. This differential stiffness, which will be henceforth referred to as the bearing stiffness, can be converted into a linearly distributed stiffness, k_s, for a given model by multiplying it by the diameter of the peg in the model. It should be noted that k_s values represent the effective bearing stiffness of the interface between the peg and the member. Thus, they include the effective bearing stiffnesses of both the peg and base materials acting in series.

Schmidt and Daniels (1999) made efforts specifically to model and predict the bearing stiffness in wood peg joints. They proposed the use of a spring-in-series model to represent the combined effect of deformation of the base material and peg material simultaneously. In theory, one could then find the bearing stiffness of each experimentally, using a relatively non-deformable solid, such as steel, and then combine the values to obtain a bearing stiffness that represents the combined interface. While the theory seems sound, the results were not conclusive and the model yielded stiffnesses around 25% lower than those found experimentally. More work would need to be done to assess the validity of the model or to understand the mechanics that render it inaccurate.

Since the validity of the spring-in-series model is in doubt, it will be necessary to use experimental results from combined bearing tests that incorporate both wooden pegs and base material. Since both materials affect the stiffness, it may be necessary to find tests that use the same materials as the model that is being analyzed. Literature on bearing tests with wooden pegs is relatively limited, and tests using the desired materials will be even more so.

In an effort to validate the spring-in-series model described above, Schmidt and Daniels performed a number of combined tests with peg and base material. They loaded 1″ diameter White Oak pegs in Red Oak base material 1.5″ thick and found an average stiffness is 69100 lb/in, giving a bearing stiffness of 47500 lb/in/in^2 with a standard deviation of 6800 lb/in/in^2.

While investigating bearing strength, Schmidt and MacKay (1997) also found average stiffness values for 1″ diameter Red Oak pegs in two different base materials as shown in Table 5.4. Orientation of peg and base material were both varied and the effect of base material orientation is shown. The effect of peg orientation was considered to be small, so overall averages are shown. Base blocks were 2*D* wide, and bearing stiffnesses were calculated accordingly.

Table 5.4 - Bearing stiffness values from Schmidt and MacKay (1997)

Base Material	Base Orient.	Avg. Stiffness (lb/in)	Avg. Bearing Stiffness (lb/in/in^2)	Bearing Stiffness Standard Deviation (lb/in/in^2)
Recycled Douglas Fir	LT	63982	31595	8455
Recycled Douglas Fir	RT	35102	17344	4843
Eastern White Pine	LT	62120	30732	4253
Eastern White Pine	RT	24977	12354	2495

For both of the experimental series above, tests consisted of compressing pegs between a block of base material with a semi-circular trough and a steel bearing plate, as shown in Figure 5.21. Thus, there are three possible locations of deformation: crushing of base material at peg, crushing of peg at base material, and crushing of peg at steel base plate. Ideally, one would only want to include the crushing of one base material-peg interface (represented by the first two deformations) but it is not clear how such an experiment could effectively be performed. Therefore, the values presented above will be used, but it is noted that the resulting stiffness values may be lower than in reality due to the fact that there are three springs in series as opposed to only two.

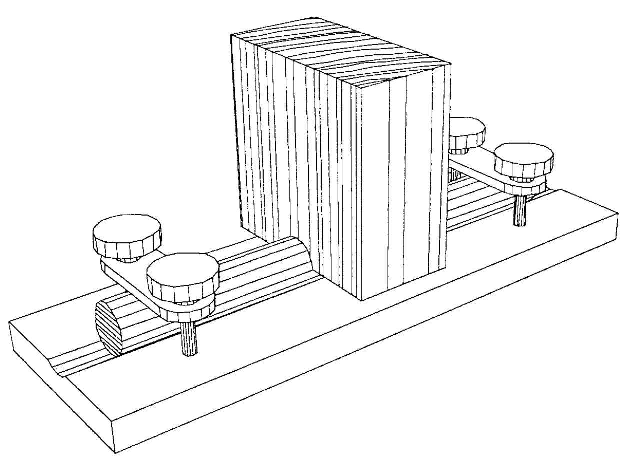


Figure 5.21 - Combined Bearing Test Setup from Schmidt and MacKay (1997)

McFarland-Johnson attempted to perform several peg bearing tests by crushing pegs between two blocks of Eastern White Spruce. Four tests were performed each with slightly different setups. The test details and results are presented in Table 5.5. Bearing stiffness was calculated by assuming crushing on both sides of the peg and dividing by the bearing area (3" width x peg diameter).

Table 5.5 - Bearing stiffness values from McFarland-Johnson (1995)

Peg Species	Diameter	Peg Alignment	Member Alignment	Test Stiffness (lbs/in)	Bearing Stiffness (psi/in)
Northern Red Oak	1 3/4"	Parallel	Perpendicular	28800	11000
Northern Red Oak	1 3/4"	Parallel	Parallel	41900	16000
White Oak	2"	Perpendicular	Perpendicular	16800	5600
White Oak	2"	Perpendicular	Parallel	34500	11500

In addition to the lack of repetition in the tests, the bearing stiffness results from the McFarland-Johnson report were acknowledged to be questionable as the tests were stopped at relatively low loads due to contact between the loading blocks. It was suggested that the stiffness values could actually increase as the peg and grain became more fully engaged.

It is noted that in the results from Schmidt and MacKay and McFarland-Johnson there is a significant difference in bearing stiffness when the base material orientation is changed. In mortise and tenon experiments, the mortise and tenon materials are oriented perpendicular to each other. Thus, it would be ideal to use varying spring stiffnesses along different sections of the peg. However, this is not feasible for this particular model. Thus, where necessary, the results for each bearing stiffness value will be presented and then compared.

5.2.4.1 - Model Data

<u>Model 1 - (Sandberg et al. 2000)</u>
The Sandberg tests were performed with Red Oak pegs in either Eastern White Pine (EWP) or Sugar Maple, with grain perpendicular to load in the mortise members and grain parallel to load in the tenon members. Modulus of elasticity can be calculated based on specific gravities. Focusing on the results for Eastern White Pine, several of the average bearing stiffnesses from Schmidt and MacKay can be used. Low bearing stiffness will be taken from loading perpendicular to grain and high bearing stiffness will be taken from loading parallel to grain.

<u>Model 2 - (Burnett et al. 2003)</u>
The Burnett tests were performed with Red Oak pegs in Eastern White Pine, Douglas Fir (DF), and Red Oak, with grain perpendicular to load in the mortise members and grain parallel to load in the tenon members. Modulus of elasticity can be calculated based on specific gravities. Focusing on the results for Eastern White Pine and Douglas Fir, several of the average bearing stiffnesses from Schmidt and MacKay could be used.

Low bearing stiffness will be taken from loading perpendicular to grain and high bearing stiffness will be taken from loading parallel to grain.

Model 3 - (McFarland-Johnson 1995)
The McFarland-Johnson tests were performed with White Oak pegs in Eastern White Spruce (EWS), with grain perpendicular to load in both the mortise and tenon members. Modulus of elasticity values are taken from the *Wood Handbook* (Forest Products Laboratory 1999). For bearing stiffness, results from McFarland-Johnson are the most applicable material-wise, however the question of validity from the McFarland-Johnson report must be considered. Since Eastern White Spruce is likely to have values similar to Eastern White Pine based on standard properties from the *Wood Handbook*, an average bearing stiffness from Schmidt and MacKay will be used for comparison. Low bearing stiffness will be taken from McFarland-Johnson (1995) for Eastern White Spruce loaded parallel to grain and high bearing stiffness will be taken from Schmidt and MacKay (1997) for Eastern White Pine loaded parallel to grain.

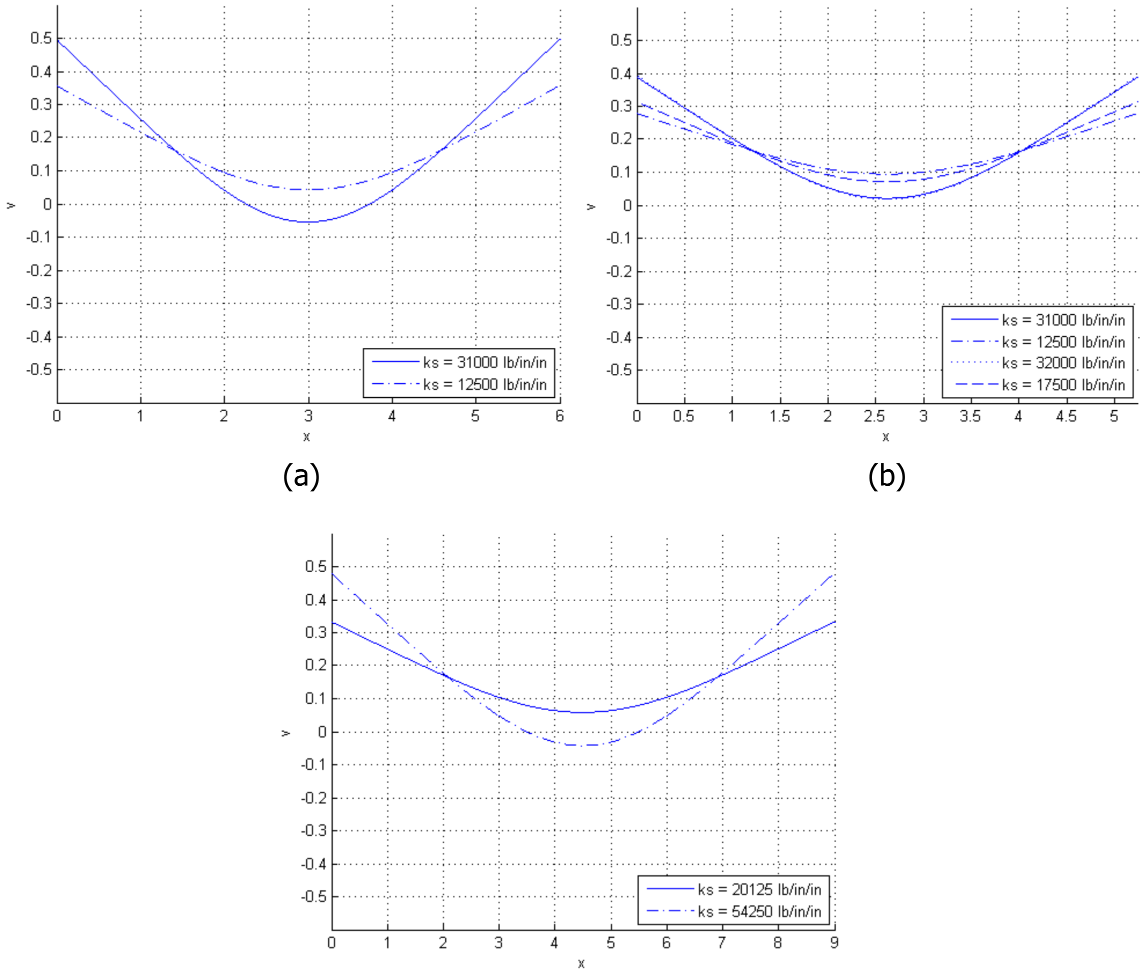
A summary of the numerical values to be used for all three experimental models is presented in the Table 5.6.

Table 5.6 - Summary of numerical values used for stiffness models

		Model 1 EWP	Model 2 EWP	Model 2 DF	Model 3 EWS
L_1	(in)	2	1.75	1.75	3
L_2	(in)	2	1.75	1.75	3
D	(in)	1	1	1	1.75
I	(in^4)	0.049	0.049	0.049	0.46
G_{12}		0.615	0.75	0.75	
E	(10^6 psi)	1.7	1.95	1.95	1.78
k_s low	(lb/in/in)	12500	12500	17500	20125
k_s high	(lb/in/in)	31000	31000	32000	54250
K avg	(lb/in)	35900	13300	21000	60300
K stdev	(lb/in)	4160	~1000	~1250	10600

5.2.5 - Results

Analytical results were found for all three models. Resulting peg shapes for all three models are given in Figure 5.22 and numerical experimental and analytical results for stiffness are given in Table 5.7.



(a) (b) (c)

Figure 5.22 - Results for peg shape from analytical model for (a) Sandberg et al., (b) Burnett et al., and (c) McFarland-Johnson

Table 5.7 - Summary of numerical values used for stiffness models

		Model 1 EWP	Model 2 EWP	Model 2 DF	Model 3 EWS
$\tilde{K}$ low	(lb/in)	14011	13225	17915	34668
$\tilde{K}$ high	(lb/in)	29670	29411	30208	79535
K avg	(lb/in)	35900	13300	21000	60300
K stdev	(lb/in)	4160	~1000	~1250	10600

5.2.5.1 - Discussion of Analytical Results

It can be seen from the plots of shape for each of the models, shown in Figure 5.22, that the predictions for shape are similar throughout. In all cases, a large amount of the deformation is taken by the base material. This can be seen by in the shape plots by considering that the neutral location is $v = 0.5$ in the mortise and $v = -0.5$ within the tenon and noting that the peg is significantly separated from these neutral locations. Another common feature of the shapes is a lack of inflection points. While the curvature decreases as the peg approaches its end, it never reverses direction. In all cases, the stiffness of the base material is too small relative to the peg bending rigidity to create a restoring moment within the mortise.

Numerical results show a less than perfect prediction of stiffness. In two of the four cases, the experimental stiffness is contained between the high and low predicted stiffnesses, as would be expected. In the other two cases, however, the experimental stiffness is close to or beyond the extreme of the predicted range, in one at the lower end of the range, and in the other at the upper end of the range.

While the analytical results are not exact matches for the experimental results, there is enough similarity for the model to be a useful estimation tool, and it represents a more accurate representation than any other models found in the literature.

One of the most important factors limiting improvement of this model is the lack of available data, specifically in terms of both bearing stiffness for varying wood species and peg diameters and overall joint stiffness for a variety of configurations.

One likely reason for some of the discrepancy between the analytical and experimental results is differences in moisture content. Moisture content is a crucial factor in experimental work involving wood, as moisture content can have a significant impact on the mechanical properties of the wood.

Average moisture content values from literature used in the work are:

Schmidt and MacKay	12% for all materials
Sandberg	6% for Eastern White Pine members 8% for Sugar Maple members 8% for pegs
Burnett	11% for all softwood members 9.8% for pegs
McFarland-Johnson	12-14% for Eastern White Spruce members 8-10% for White Oak pegs

Typically, modulus of elasticity increases as moisture content decreases, and it is expected that bearing stiffness will do likewise. The bearing stiffness values used in the analytical models are largely based on the research by Schmidt and MacKay, which used a moisture content of 12%. This could explain why the predicted stiffnesses for model 1 are lower than the experimental values, which come from research that used material with a moisture content of 6%. More research would need to be performed to assess the effect of moisture content on joint stiffness.

5.2.5.2 - General Results

With the goal of using the analytical model as an estimation tool for connection stiffness, some general results can be extracted from the formulation. *K/EI* can be plotted as a function of λ for different member thicknesses, as shown in Figure 5.23.

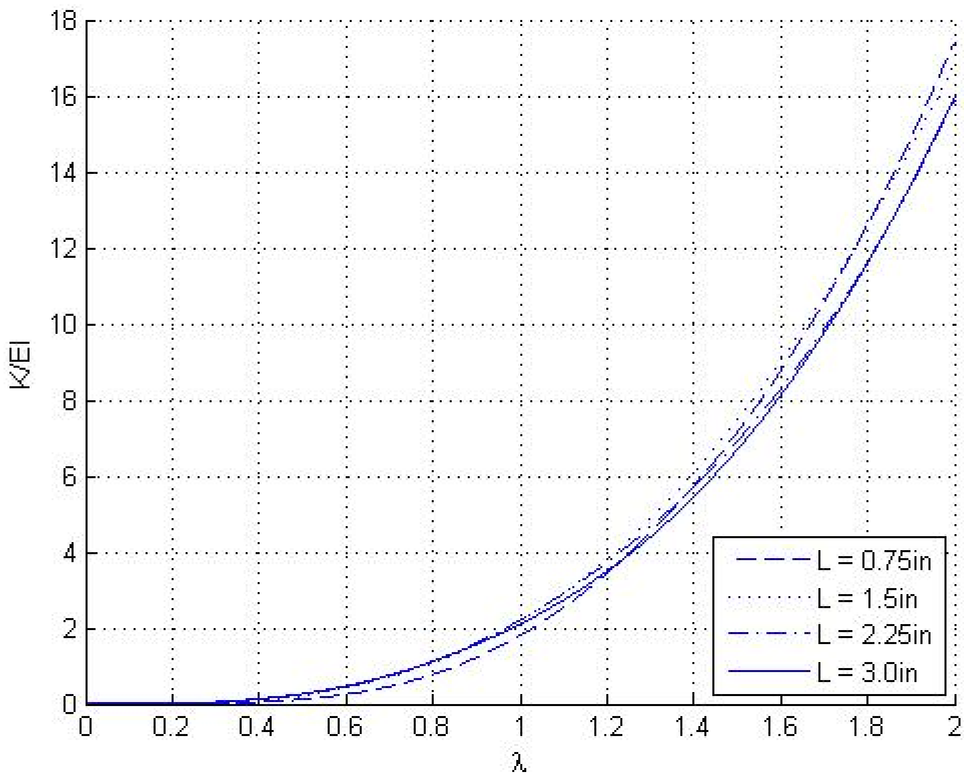


Figure 5.23 - Plot of *K/EI* versus λ for Double Shear

The results used to plot the curves above cannot be easily converted into simple equations, which are desirable for use in estimation for design. To solve this, a regression analysis is performed to determine the best-fit curve for each of the member thcknesses over a reduced range of λ = 0.1 to 0.75, which is thought to contain all of the results likely to be seen in the design of Town lattice truss connections. Results of the regression analysis are shown in Figure 5.24.

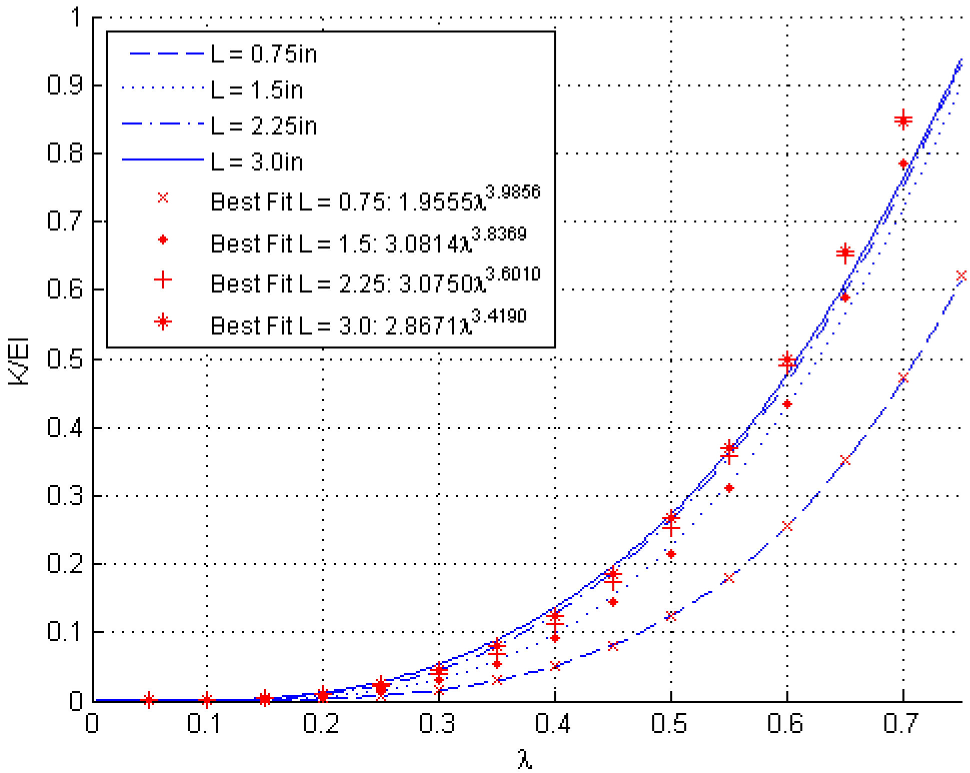


Figure 5.24 - Regression results for stiffness prediction curves for double shear wooden pegged connections

The best-fit equations shown in Figure 5.24 can be reduced down to base material and geometric properties. For L = 3.0 in, the best fit equations is:

$$\frac{K}{EI} = 2.867 \cdot \lambda^{3.4190}$$

$$K = 2.867 \cdot EI \cdot \left(\sqrt[4]{\frac{k_s}{4 \cdot EI}} \right)^{3.4190}$$

$$K = \frac{2.867}{4^{0.8548}} \cdot \frac{EI}{(EI)^{0.8548}} \cdot (D \cdot k_b)^{0.8548}$$

$$K = 0.8766 \cdot \left(\frac{1}{64} \cdot \pi \cdot D^4 \right)^{0.1453} \cdot E^{0.1453} \cdot D^{0.8548} \cdot k_b{}^{0.8548}$$

$$K = 0.5658 \cdot E^{0.1453} \cdot D^{1.436} \cdot k_b{}^{0.8548}$$

The other equations can be reduced similarly, yielding a final set of equations for use in the prediction of stiffness for double-shear pegged connections.

$K = 0.4860 \cdot E^{0.0036} \cdot D^{1.011} \cdot k_b{}^{0.9964}$ for L = 0.75 in

$K = 0.7209 \cdot E^{0.0418} \cdot D^{1.122} \cdot k_b{}^{0.9592}$ for L = 1.5 in

$K = 0.6535 \cdot E^{0.0998} \cdot D^{1.299} \cdot k_b{}^{0.9003}$ for L = 2.25 in

$K = 0.5658 \cdot E^{0.1453} \cdot D^{1.436} \cdot k_b{}^{0.8548}$ for L = 3.0 in

5.2.6 - Stiffness results

The goal of the preceding analyses was to develop a model for the estimation of the stiffness of Town lattice truss connections. As discussed previously, the Town lattice truss connection is conventionally considered to be equivalent to half of a double shear connection. Therefore, the stiffness estimation equations that have been developed must divided in half to yield the stiffness of a single peg in a Town lattice truss connection. To get the full stiffness of a Town lattice truss connections, the stiffness of a single peg must be multiplied by the number of pegs in the connection. For this work, only 3” thick members will be used, yielding the equation:

$$K = N_{peg} \cdot 0.2829 \cdot E^{0.1453} \cdot D^{1.436} \cdot k_b^{\ 0.8548} \text{ for } L = 3.0 \text{ in}$$

5.3 - Summary

The strength and stiffness of the components of a structure will have an impact on the overall behaviour of the structure. The Town lattice truss has unique wooden pegged connections for which there is little experimental data and no established procedures for the determination of strength and stiffness.

Equations to determine the design strength of wooden pegged connections were developed based on models and experimental data from a variety of sources. These equations represent the best strength prediction possible at the current time, though there is a certain level of uncertainty due to a lack of experimental data, particularly in the wooden dowel bearing and shear failure modes that were found to dominate in all relevant situations.

An estimation equation was developed for the stiffness of the connections based on a beam on elastic foundation analytical model. The model offers reasonable shape results that are consistent with experimental behaviour but numerical results are inconsistent. High material variability and limited experimental data make it impossible to improve the model at this time, and the results are considered reasonable for use as an estimation tool in designing the Town lattice truss. Further experimental work and modeling is recommended to assess the validity of the beam of elastic foundation model for use in stiffness prediction.

The connection properties will have an impact on the behaviour of the chords of the Town lattice truss, and the following chapter will address methods of combining member and connection properties to determine overall chord properties.

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Chapter 6 – The Town Lattice Truss: Chords

Chord termination patterns are a unique element of the Town lattice truss and another characteristic that contributes to the appropriateness of the truss. The staggering of chord terminations and distribution of forces through a large number of connections helps to reduce the need for skilled carpentry work in the construction while maintaining a strong structural system. However, this unique aspect of the Town lattice truss has never been studied and the effect of chord termination patterns on the properties of the chords is not understood.

Individual chord patterns have been analyzed when assessing a specific bridge. Finite element packages are typically used and the results will be applicable to that bridge only. In the design of new bridges, it is important to be able to select the pattern to be used based on an understanding of the effect this pattern will have on the strength and stiffness of the truss.

In this chapter, chord termination patterns are studied by first identifying a comprehensive set of patterns that could be selected for use in the Town lattice truss. All possible patterns must have a minimum level of structural integrity and must be unique from all other patterns.

The list of all possible patterns is then analyzed to determine maximum design strength based on sets of possible failure modes. Design strengths will be based on combinations of design strengths for members and connections, as identified in chapter 5.

The list of possible patterns will also be analyzed to determine an effective stiffness factor, which will have an impact on deformations in the final bridge and may have an impact on the distribution of force within the cross-section of the bridge when subjected to bending moment.

Finally, patterns are recommended for use in the design of Town lattice trusses based on the results of the strength and stiffness analysis. The strength and stiffness properties of chords using these patterns are summarized in a way that is simple to apply in the design of the trusses.

6.1 - Development of patterns

Many different chord termination patterns were seen in the study of Town lattice truss bridges in the northeastern United States. No clear rationale behind the selection of patterns seems to exist. It is desirable to have a scheme for comparing patterns based on engineering knowledge.

To be able to compare patterns, and potentially select the best patterns, it is important to first determine all possible patterns that could be used. A systematic comprehensive approach to pattern identification will be used.

6.1.1 - Description of pattern lengths

For the purposes of this analysis, it is assumed that all members used in the construction of the chords of the lattice will be the same length, with the exception of end members which may be cut shorter due to the termination of the truss. This assumption is reasonable for the Town lattice truss, for which repetition of members and construction is a key advantage.

As an additional constraint, it is assumed that all chord members will have a length that is equal to an integer number of joint spacings, henceforth referred to as units. This is somewhat more constraining than the above equal length assumption, but is essential in the Town lattice truss to ensure that all chord terminations occur between lattice intersections. The presence of a chord termination at the same location as a pegged connection will have implications on the strength and stiffness of the connection.

If all members are the same length, all four lines of chord members must have a chord termination within each longitudinal chord section of that length. The next chord termination for each line will occur one member length further along the chord. Thus, there will be a repeating pattern of chord terminations that has a length equal to the length of the members used in the chords. Since the members are an integer number of units long, the length of the pattern will also be defined by this integer number. This relationship is illustrated in Figure 6.1.

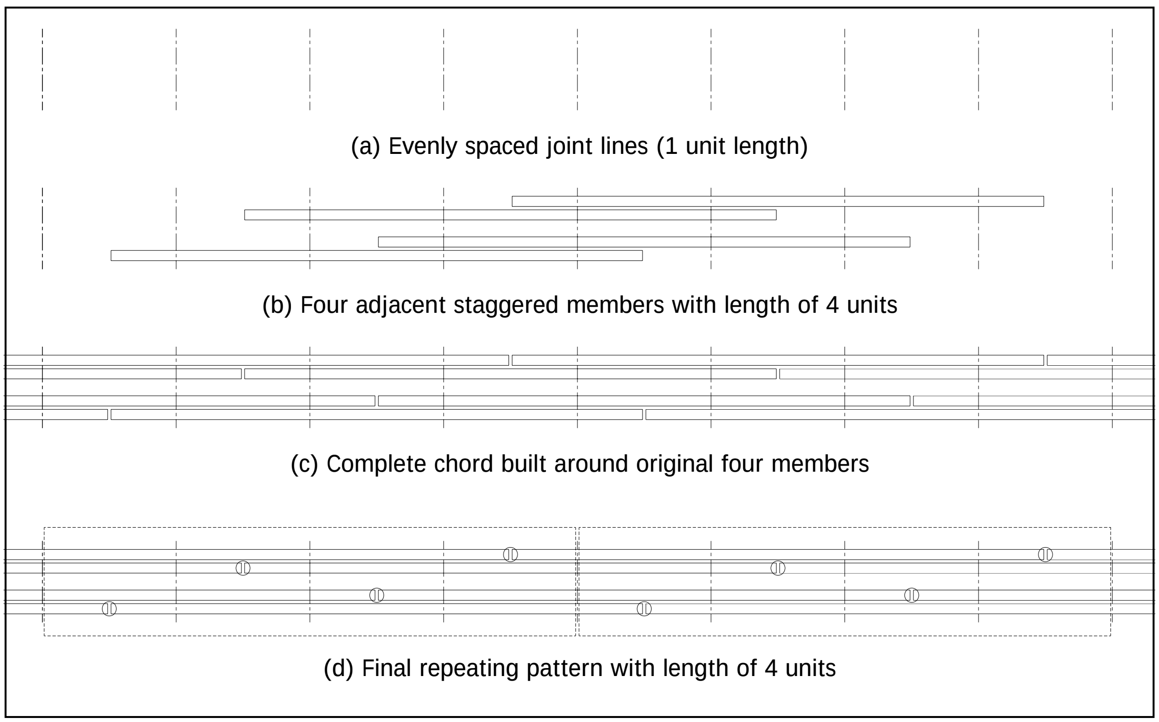


Figure 6.1 - Development of 4-unit pattern from 4-unit members

Patterns seen in Town lattice truss bridges ranged from 4 to 9 units long, although only a single bridge had members 9 units long and is considered atypical. For this work, patterns from 4 to 8 units long will be considered.

6.1.2 - Basic rules of valid patterns

There are two primary requirements used to determine all of the possible patterns for use in the Town lattice truss bridge. First, the chord of the Town lattice truss must behave in a structural manner, and it is essential that the patterns allow for a minimum level of structural capacity. To ensure this, it is first required that there be no location with more than two chord lines with terminations. This ensures a minimum of two solid members at any cross sectional cut through the chord. In addition, it is required that at no point should either outside pair of chords have both members with terminations at the same location. This ensures that there is always a mechanism to carry load on either side of the web in the chord structure. Patterns that do not meet this requirement will have an unbalanced load capacity that is not desirable. The second requirement will inherently satisfy the first, since it is impossible to have three chord terminations at the same location without two of them being in one of the outside pairs of chords.

The resulting requirement for there to be at least one unbroken chord member on each side of the web at all locations will be henceforth referred to as the "minimum structural integrity" requirement.

The second requirement for a valid pattern is that it must be unique. In recording a pattern of chord terminations in a Town lattice truss bridge, there are a number of arbitrary choices that will define how the pattern is identified. For example, the starting point of the pattern will affect all of the locations within the pattern, though not relative to each other. Similarly, the choice of whether to denote the inside or the outside line of chord members first will affect the order of the locations. Finally, the choice of which direction along the chord is indicated by location 1 through N will change the resulting numbering. Despite the differences in numbering, none of these choices will change the actual physical pattern of the chord terminations. Examples of different numbering sequences being derived from the same chord are shown in Figure 6.2.

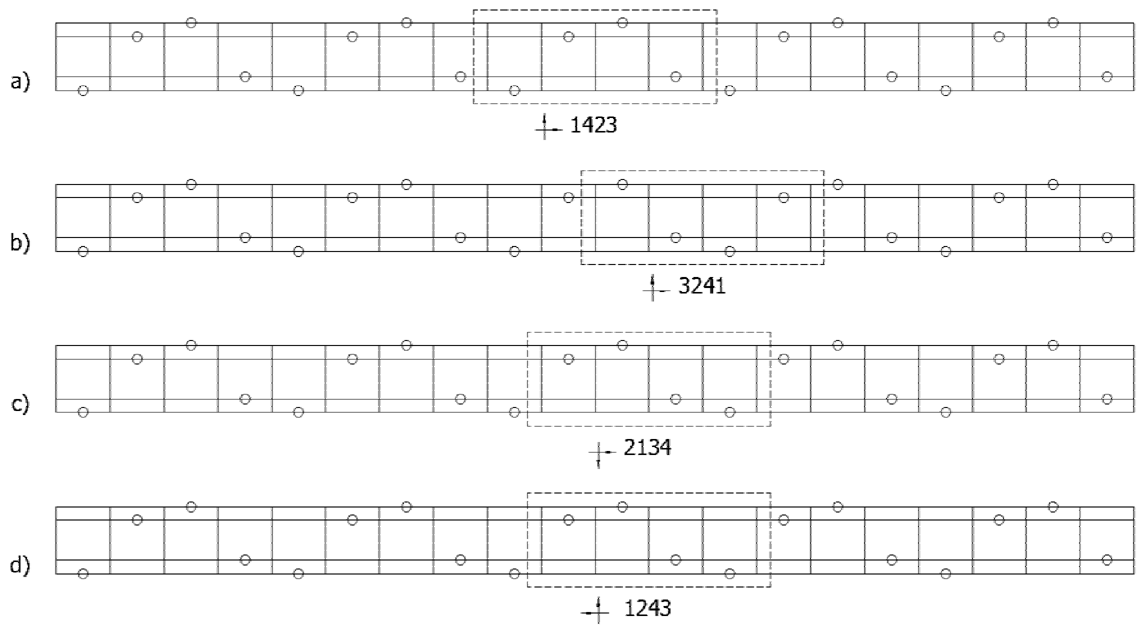


Figure 6.2 - Example of multiple patterns numberings being derived from the same chord a) standard, b) translated, c) translated and mirrored transversely, and d) translated and chord direction reversed

In a similar manner, two patterns with the same numbering scheme could in reality run in opposite directions or have different starting points within the truss. However, it is not expected that these small differences in starting location and direction of pattern will have any significant structural effect. In fact, it would be impractical in the design of a bridge to mandate an exact starting location and direction for a chord pattern, and thus it is desirable to consider all matching patterns to be equivalent.

There are four types of derivatives of any given pattern that will be defined for use in comparing patterns for uniqueness. These are: translated patterns, mirrored patterns, reversed patterns, and mirrored-reversed patterns.

6.1.2.1 - Translated patterns

A translated pattern is one that is different from the original as a result of a different starting position. Any pattern number can be translated by adding an integer to, or subtracting an integer from, the location of the termination for all lines of chord members. Any location that ends up greater than the length, N, or less than 1 should have the length of the pattern subtracted from it or added to it as appropriate. It is possible to derive N different translated patterns for any given pattern of length N. For example, 4-unit pattern 1423, as used as the base in Figure 6.2, yields the derived translated patterns of 4312, 3241, and 2134, as illustrated in Figure 6.3.

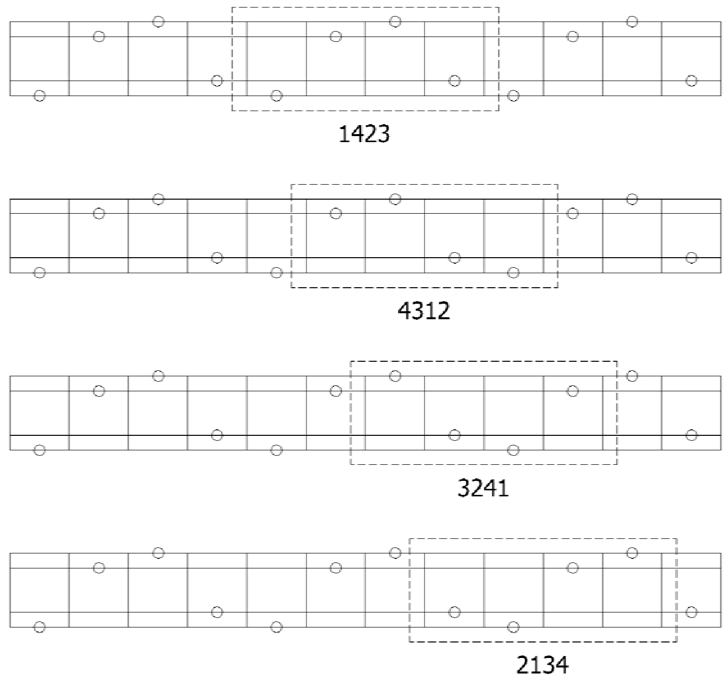


Figure 6.3 – Example of derived translated patterns

Translation will be accounted for in comparing patterns by adjusting all patterns to have the termination of the first line of chord members in location 1 before comparison.

6.1.2.2 - Mirrored patterns

A mirrored pattern is one that is different from the original as a result of reversing the order of the lines of chord members. Thus, the pattern is mirrored about a longitudinal line extending down the centre of the chord structure. The numbering of a mirrored pattern can be derived from the original by reversing the order of the numbers. For example, 1423 yields the mirrored pattern 3241, as illustrated in Figure 6.4.

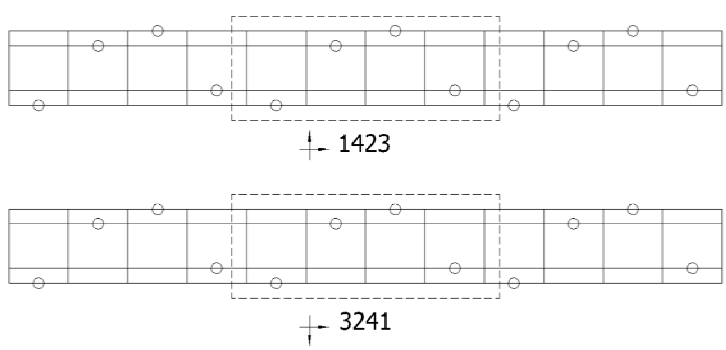


Figure 6.4 - Example of a derived mirrored pattern

6.1.2.3 - Reversed patterns

A reversed pattern is one that is different from the original as a result of reversing the location numbers 1 through *N* to be *N* through 1. Thus, the chord termination locations are counted in the opposite direction along the chord. The numbering of a mirrored pattern can be derived from the original by subtracting *N* + 1 from all locations and taking the absolute value of the result. For example, pattern 1423 yields the reversed pattern 4132, as illustrated in Figure 6.5.

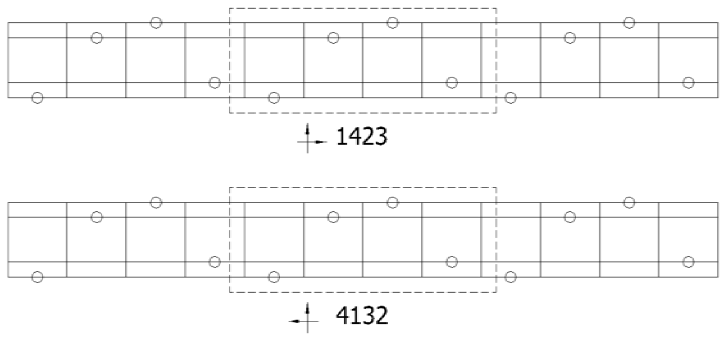


Figure 6.5 - Example of a derived reversed pattern

6.1.2.4 - Mirrored-Reversed patterns
A mirrored-reversed pattern is one that is different from the original as a result of both reversing the order of the lines of chord members and reversing the location numbers 1 through *N* to be *N* through 1. The numbering of a mirrored-reversed pattern can be determined by either performing a mirroring of the original's reversed pattern, or reversing the original's mirrored pattern. For example, pattern 1423 yields the mirrored-reversed pattern 2314, as illustrated in Figure 6.6.

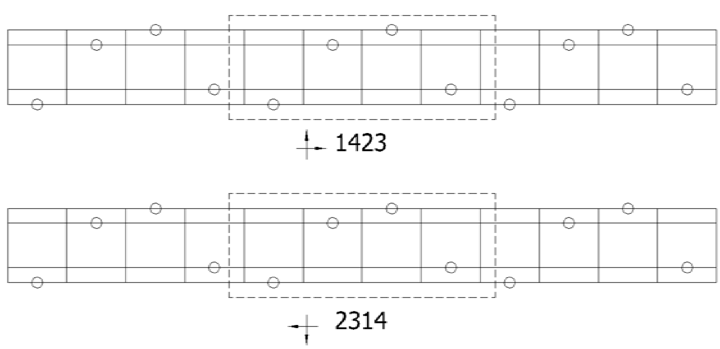


Figure 6.6 - Example of a derived mirrored-reversed pattern

6.1.3 - Description of pattern making procedure

The goal in developing patterns is to be complete and include all possible patterns. Thus, a systematic approach is employed. At the most general level, there are 4 lines of chord members, each of which must have a termination within the repeated unit length. Therefore, any pattern can be represented by five numbers. The first number, *N*, represents the length of the pattern in units or joint spacings. The second number represents the location of the termination for the first line of chord members, which must be between 1 and the length of the pattern, *N*. The third number similarly represents the location of the termination in the second line of chord members, and so on.

To generate all possible chord termination patterns for a given length, one must simply step through all permutations of four numbers with *N* choices. This is done with four nested incrementing loops. The resulting possible patterns must then all be checked for validity and uniqueness, keeping only those that meet both criteria.

Validity is determined first by checking if the pattern meets the minimum structural integrity requirement. If it does not, it is eliminated. If it does, it is next be checked for uniqueness.

To check for uniqueness, a valid possible pattern is compared with all recorded patterns as well as each of their derivative patterns, with all compared patterns having been translated such that the first termination of each is in location 1. If any matches are found, the new pattern is eliminated. If no matches are found, the pattern is recorded along with all three of its derivative patterns.

An example of this process will now be described. For units of length 4, the permutation will start by producing 1111, 1112, 1113, 1114, 1121, etc… Since the minimum structural integrity requirement states that no two outside chords can have breaks in the same location, the first pattern that will be accepted as valid is 1212. Since this is the first pattern, it is inherently unique and is recorded along with its derivatives – mirrored 2121, reversed 4343, and mirrored-reversed 3434. All four patterns are shown in Figure 6.7

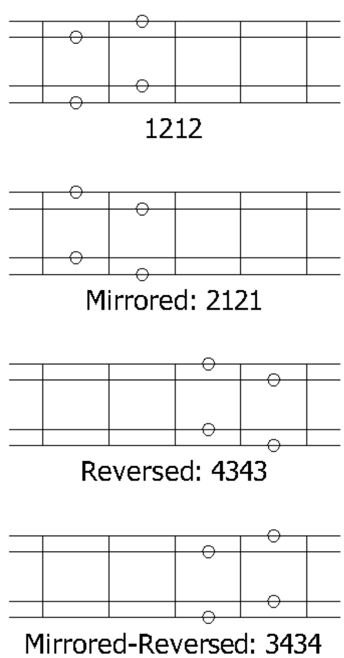


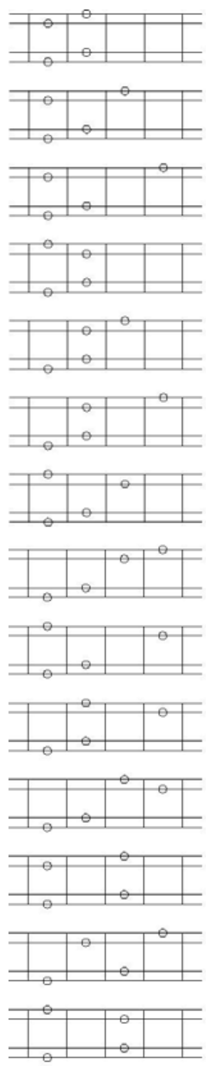
Figure 6.7 - Pattern 1212 and derivative patterns

The process then continues with the next permutation, 1213, which is also a valid and unique pattern and will be recorded. The process continues and at some point a non-unique pattern will be found and eliminated. An example of a pattern that conflicts with 1212 is 1414.

At first glance it may not seem as if pattern 1414 will match with 1212. A direct comparison with pattern 1212 and all of its derivatives yields no exact match. However, it is necessary to translate all patterns to the same starting location before comparison. The reverse and mirror of 1212 both become 1414 when translated such that their first termination is in location 1. This produces a match and identifies 1414 as non-unique.

6.1.4 - Pattern results

A full set of patterns for 4-unit patterns is shown in Figure 6.8. Complete results for patterns from 4-units to 8-units long are given in Appendix E.



1	2	1	2
1	2	1	3
1	2	1	4
1	2	2	1
1	2	2	3
1	2	2	4
1	2	3	1
1	2	3	4
1	2	4	1
1	2	4	2
1	2	4	3
1	3	1	3
1	3	2	4
1	3	3	1

Figure 6.8 - All valid and unique 4-unit patterns

6.2 - Strength of patterns

The strength of a pattern will be based on its potential failure mechanisms. In tension, the chord terminations will separate and load will be carried by a combination of chord members and pegged connections. For the chord to fail in tension, there must be a mechanism for a complete failure plane through the width of the chord. Since the patterns result in an interleaving of chord members, the failure plane need not exist at a single transverse location and may be comprised of a combination of cuts through members and pegged connections.

A given pattern can yield a multitude of potential failure paths, however it is only the ones with minimum strength that are of interest. Without specific values for connection and member strength, it is not possible to determine the absolute minimum failure mechanism for a pattern, but it is possible to narrow the possible failure mechanisms down to a limited few, which will allow for comparison between patterns.

6.2.1 - Component strength

The overall strength of a pattern will be based on the sum of the strengths of the components that make up the chord structure, namely the pegged connections and the chord members themselves. It is important to understand both the capacity of the component and the nature of the failure that will occur, as this will define how the strengths are combined.

6.2.1.1 - Pegged Connection Failure Values

The failure of pegged connections is conventionally considered to occur at "yield strength". For the wooden components of the connection, "yield strength" is defined by the 5% offset method. Since the strength of the connections is based on yield modes, and is determined by the underlying components, overall connection strength will also be based on yield strength from the 5% offset method. This method was presented in Chapter 5 and is described in ASTM D5652 – Standard Test Methods for Bolted Connections in Wood and Wood-Based Products (ASTM 2000).

The 5% offset method defines the yield load seen in an experimental test as the load at the intersection of the experimental load-displacement curve and a straight line parallel to the linear portion of the curve and offset from it by a distance of 5% of the peg diameter D, or 0.05D. If the load-displacement curve does not intersect the offset line, or if the intersection load is less than the maximum load seen in the test, the yield load is taken as the maximum load in the test.

The resulting yield strength does not actually represent a clear yield point, since wood does not follow a bi-linear relationship, but is indicative of a point where larger deformations begin to occur. Most connections have at least some capacity beyond the yield load, although this is not always the case.

The design yield strength of a pegged connection is calculated as the sum of the design yield strength of each of the pegs in the connection. The design yield strength of pegged connection can be calculated using strength equations developed in Chapter 5. The final equations to be used to determine the strength of a pegged connection in a Town lattice truss are:

Mode $\mathbf{I_d}$ $\quad ZI_d = 662.5 \cdot D \cdot t \cdot G_{dd}^{2.04}$ or $ZI_d = \dfrac{D \cdot t \cdot F_{ed}}{8}$

Mode $\mathbf{V_d}$ $\quad ZV_d = 538.8 \cdot D^2 \cdot G_{dd}^{0.84}$ or $ZV_d = 0.223 \cdot D^2 \cdot F_{ev}$

These equations are determined as half of the predicted strength of a double shear connection, further reduced by the load duration factor and an appropriate safety factor. In the case that strength properties are available for dowel bearing and dowel shear, they should be used with the second equation for each mode. If this experimental data is not available, the dowel specific gravity, either estimated or measured, can be used in the first equation for each mode to provide a prediction of strength.

6.2.1.2 - Member Failure Values

Member failure will occur in tension at the smallest cross-section of a member. This smallest cross section will occur at the location of the pegged connections, which will generally remove two diameters worth of depth from the overall depth of the chord members. The design failure load of the member can then be calculated as the allowable stress in tension multiplied by this reduced cross-sectional area.

Tension failure in wood is generally brittle in nature and is greatly affected by defects in the member. As a result of these two factors, allowable stress values in tension for structural members are significantly reduced from the theoretical tension capacity of wood.

Allowable stress values for specific wood species should be used to determine the design capacity of the chord members. Load duration should be considered, and appropriate factors should be applied to the allowable stress.

Because tension failure is generally brittle in nature, it is not assumed that there will be further capacity beyond the failure load.

6.2.2 - Determination of failure planes

In a similar manner to the determination of patterns, a systematic approach is taken in the determination of possible failure planes. Failure planes can range from simple, such as a cut straight through the chord section, to complicated, interleaving between all members. These two extremes are defined by a failure plane that cuts only through members and a failure plane that cuts only through pegged connections. Between these two extremes are failure planes that incorporate a combination of member and connection failures.

The failure plane represents a location where the chord separates into two completely distinct pieces. For failure to occur, all of the components that intersect the failure plane must reach their respective failure values. Therefore, the overall capacity of a failure plane will be based on a simple sum of all of the strengths of the components that intersect the plane. Since there are only two types of components, connections and members, that make up the chord, failure plane capacity can be written as:

$$F_{FP} = N_{c,FP} \cdot F_c + N_{m,FP} \cdot F_m$$

where, F_{FP} is failure plane capacity, $N_{c,FP}$ and $N_{m,FP}$ are the number of intersected connections and members, respectively, and F_c and F_m are the connection and member strengths, respectively.

6.2.2.1 - Member-only failure

A failure plane through members only must occur at a single cross-section of the chord structure. Otherwise, it must cut through a pegged connection, which will contribute to the failure strength. The minimum pure member failure will occur at the location with the most chord terminations, or conversely the location with the fewest solid members. In a pattern where all chord terminations are located at different locations, the minimum number of solid members will be three, yielding a minimum member-only failure load of $3F_m$, where F_m is the member axial strength. In a pattern where two chord terminations occur at the same location, the failure load will be $2F_m$. Since the minimum structural integrity requirement dictates that there will never be more than two chord terminations at the same location, these are the only two options for member-only failure.

Member-only failures can be determined by systematically checking all locations within a pattern and determining the number of solid members present. The cross-section with the smallest number of members will dominate. An example of this procedure is shown in Figure 6.9.

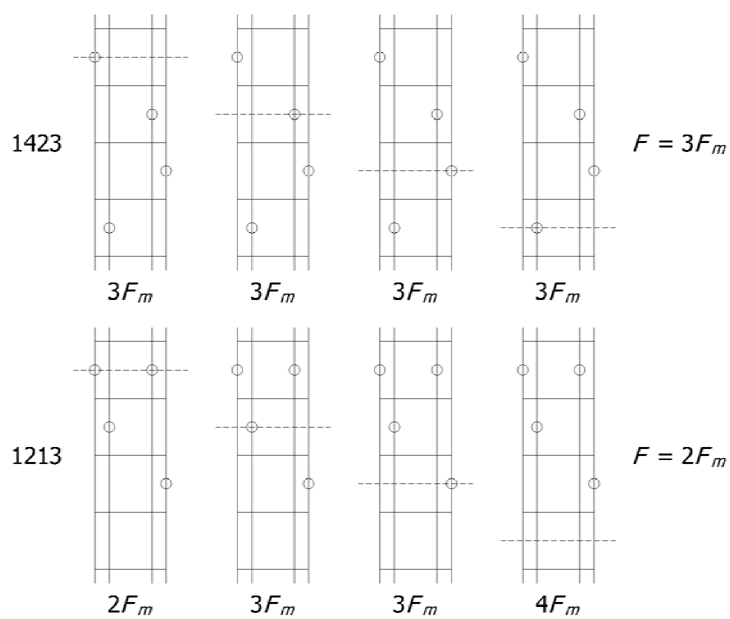


Figure 6.9 - Possible member-only failure planes for patterns 1423 and 1213

6.2.2.2 - Connection-only failure
A failure plane that only intersects connections must interleave between the lines of chord members, crossing them only at chord terminations. A finite number of possible connections-only failure planes can be developed if it is assumed that the plane will always cross a line of chord members if it encounters a chord termination. When this is assumed, all of the planes can be described by a three-bit binary number, with each bit representing a choice between the failure plane traveling up or down the chord. This being the case, there will always be eight possible planes for a given pattern. A visual representation of all possible connection-only planes through a chord section is shown in Figure 6.10. The path with the minimum number of connection intersections will dominate.

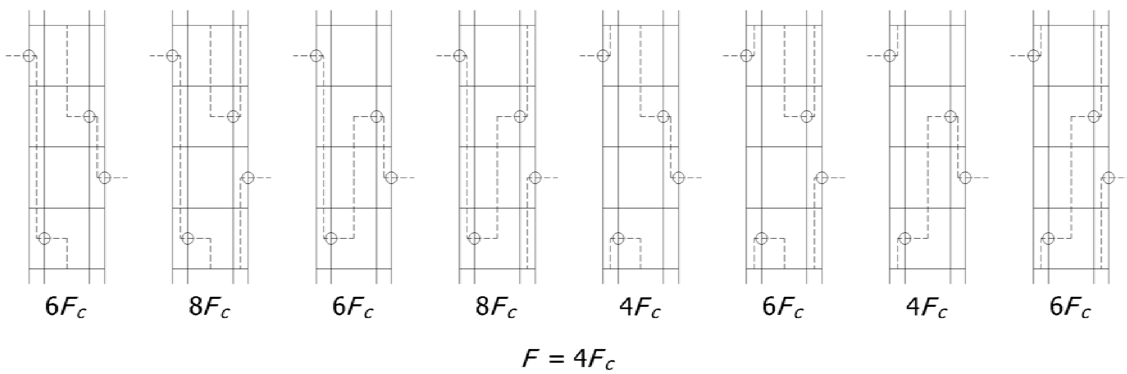


Figure 6.10 - Possible connection-only failure planes for pattern 1423

6.2.2.3 - Combination failure
A combination failure will include cuts through both members and connections. There are a multitude of possibilities for this type of failure and the vast majority of possible failure planes will yield a combination failure. It is important to be systematic and inclusive in considering possible failure planes while also limiting the output to reasonable useful values.

For a given pattern, the failure planes to check will include all permutations of the four chord locations applied over all translated derivatives of the pattern. An assumption is made that the minimum failure planes will occur within one pattern length, although that pattern length can start at any location within the chord.

The goal is to find failure planes that will have a capacity between the two extremes already defined by the member-only and connection-only failure planes. Since the connection-only failure represents the minimum failure with 0 members failing and the members-only failure represents the minimum failure with 2 or 3 members failing, it is only left to find the minimum failure with 1 member and 2 members, if needed. Figure 6.11 shows examples of 1- and 2-member combination modes for pattern 1423, along with examples of connection-only and member-only modes for the same pattern.

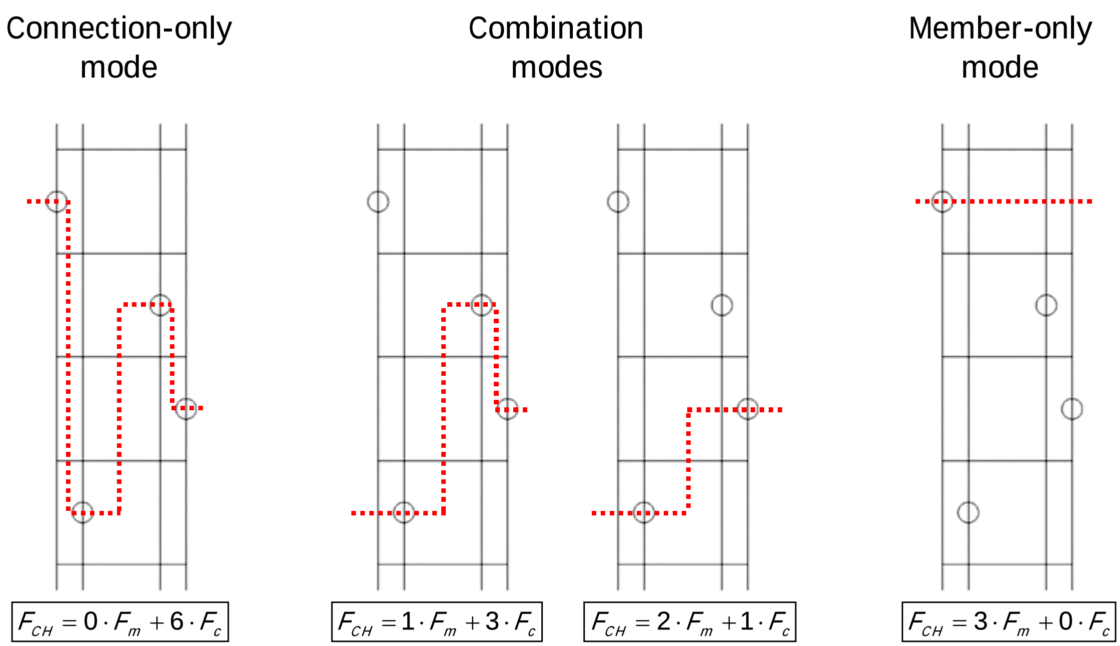


Figure 6.11 - Examples of connection-only, combination, and member-only modes

For a given permutation, the number of members is determined by counting the number of cut locations that do not correspond to chord termination locations. The number of connections is determined by summing the differences between the locations of all adjacent pairs of lines of chord members. If the resulting value has 1 or 2 members and a number of connections less than the existing minimum, it is recorded.

6.2.3 - Comparison between patterns

Without numerical values for F_c and F_m it may not be possible to draw firm conclusions about which specific pattern is the strongest. Because each pattern has failure modes from a combination of members and connections, there may be some ambiguity if the relative strengths are not known. However, it is possible to eliminate patterns for which all or most of the failure mode types have sums that are clearly less than all of the equivalent mode types of another pattern.

Failure mode results for 4-unit patterns are given in Table 6.1. Complete results for pattern failure modes are included in Appendix E.

Table 6.1 - 4-unit length patterns with failure mode capacities

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c
4	1 2 1 2	0 3	1 1	2 0	
4	1 2 1 3	0 4	1 2	2 0	
4	1 2 1 4	0 3	1 1	2 0	
4	1 2 2 1	0 2	1 1	2 0	
4	1 2 2 3	0 2	1 1	2 0	
4	1 2 2 4	0 3	1 1	2 0	
4	1 2 3 1	0 4	1 2	2 0	
4	1 2 3 4	0 3	1 2	2 1	3 0
4	1 2 4 1	0 4	1 2	2 0	
4	1 2 4 2	0 5	1 1	2 0	
4	1 2 4 3	0 4	1 2	2 1	3 0
4	1 3 1 3	0 6	1 2	2 0	
4	1 3 2 4	0 5	1 3	2 1	3 0
4	1 3 3 1	0 4	1 2	2 0	

A comparison can be conducted between any pair of patterns. An example comparison between pattern 1221 and pattern 1231 is shown in Figure 6.12 below.

Pattern

1221 1231

Connection-only mode

$$F_{CH} = 0 \cdot F_m + 2 \cdot F_c \quad < \quad F_{CH} = 0 \cdot F_m + 4 \cdot F_c$$

Combination mode

$$F_{CH} = 1 \cdot F_m + 1 \cdot F_c \quad < \quad F_{CH} = 1 \cdot F_m + 2 \cdot F_c$$

Member-only mode

$$F_{CH} = 2 \cdot F_m + 0 \cdot F_c \quad = \quad F_{CH} = 2 \cdot F_m + 0 \cdot F_c$$

Figure 6.12 - Strength comparison between 4-unit patterns 1221 and 1231

As can be seen in Figure 6.12, for any given pair of values of F_m and F_c, all of the modes for pattern 1231 have a greater strength than the equivalent modes of pattern 1221. Therefore, pattern 1231 is determined to be the preferred pattern of the two in terms of strength.

Following this modal comparison method, it is possible to approximately sort patterns based on strength. The results for 4-unit patterns are shown in Table 6.2.

Table 6.2 - 4-unit length patterns approximately sorted by strength

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c
4	1 3 1 3	0 6	1 2	2 0	
4	1 3 2 4	0 5	1 3	2 1	3 0
4	1 2 4 2	0 5	1 1	2 0	
4	1 2 4 3	0 4	1 2	2 1	3 0
4	1 2 1 3	0 4	1 2	2 0	
4	1 2 3 1	0 4	1 2	2 0	
4	1 2 4 1	0 4	1 2	2 0	
4	1 3 3 1	0 4	1 2	2 0	
4	1 2 3 4	0 3	1 2	2 1	3 0
4	1 2 1 2	0 3	1 1	2 0	
4	1 2 1 4	0 3	1 1	2 0	
4	1 2 2 4	0 3	1 1	2 0	
4	1 2 2 1	0 2	1 1	2 0	
4	1 2 2 3	0 2	1 1	2 0	

Through relative comparison between all patterns, it is possible to separate out two modes, 1313 and 1324, that each have the possibility of being the strongest mode, although it is not possible to definitively select between them without exact relative values for member and connection strength. In the case that connection-only failure dominates, 1313 will be the strongest option of all possible patterns, while in all other cases, 1324 will be as strong as, or stronger than, all possible patterns.

6.2.4 - Results and conclusions

Final best patterns based on strength are given in Table 6.3 with relevant failure modes and patterns are shown in Figure 6.13. For 4-unit patterns, as already discussed, one pattern is dominant for the connection-only mode, while a second pattern is dominant for all other modes. For 5-unit patterns, a single pattern was found to be dominant for all modes. For 6-unit patterns, similar to 4-unit patterns, one pattern is dominant for the connection-only mode, while a second pattern is dominant for all other modes. For 7-unit patterns, one pattern is dominant for the connection-only mode, a second pattern is dominant for a 1-member mode, and the two patterns are equivalent and at least as strong as all others for the 2- and 3-member modes. Finally, for the 8-unit pattern, one pattern clearly dominates for the connection-only mode, while a second pattern dominates or is equivalent for the other modes.

Table 6.3 - Highest strength patterns with lengths from 4 to 8 units

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c
4	1 3 1 3	0 6	1 2	2 0	
4	1 3 2 4	0 5	1 3	2 1	3 0
5	1 3 5 2	0 6	1 3	2 1	3 0
6	1 4 1 4	0 9	1 3	2 0	
6	1 4 2 5	0 8	1 4	2 1	3 0
7	1 4 7 3	0 9	1 4	2 1	3 0
7	1 4 6 2	0 8	1 5	2 1	3 0
8	1 5 1 5	0 12	1 4	2 0	
8	1 5 3 7	0 10	1 6	2 2	3 0

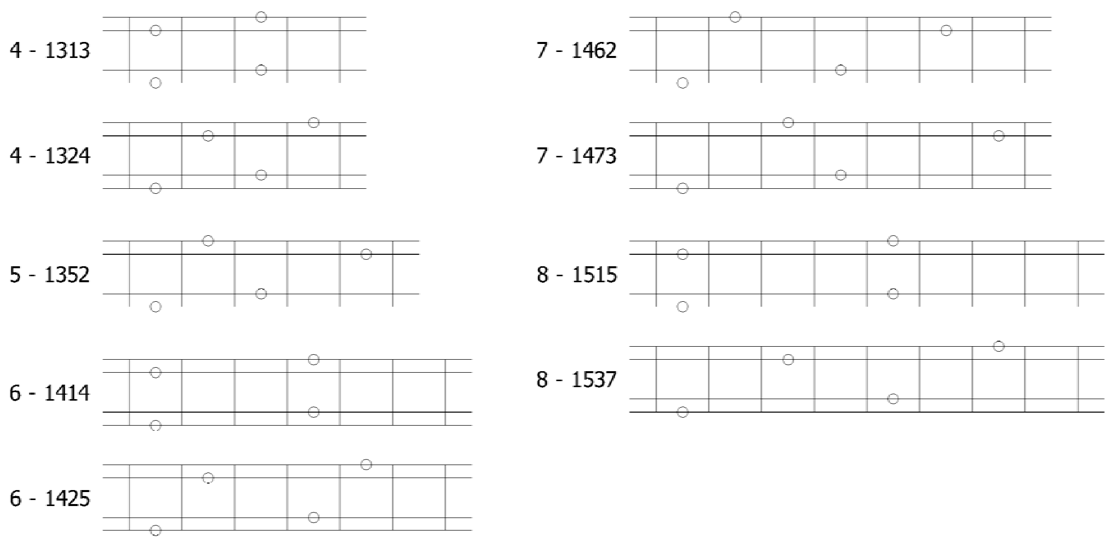


Figure 6.13 - Highest strength patterns with lengths from 4 to 8 units

Ultimately, the dominant patterns for different lengths have relatively consistent characteristics. To obtain the longest possible path in a connection-only failure, chord terminations in adjacent lines of members must be placed as far apart as possible. This can result in two pairs of collocated termination in even length patterns, such as the 6-unit 1414, or in a multiple unit diagonal pattern in odd length patterns, such as the 7-unit 1473. The patterns selected as dominating the other modes tend to be similar to those previously listed, although with one pair of outside terminations shifted slightly to eliminate collocations.

A final selection between patterns should be based on estimations of actual component strength values. In addition, the stiffness of the patterns will be assessed, as this will affect the behaviour of the truss and may offer an additional incentive to select a specific pattern if significant differences are found.

6.3 - Stiffness of patterns

The stiffness of a chord is relevant for the performance of the bridge under service loads. Too much deflection or vibration can lead to an unsafe condition for those using the bridge. Any ability to increase the stiffness of the bridge without increasing the use of materials or decreasing the bridges strength will be desirable. In addition, chord stiffness may also have an impact on the distribution of forces between chords in the Town lattice truss when subjected to a bending moment. For both of these reasons, it is worthwhile to explore the effect of chord termination patterns on the effective stiffness of the tension chords of the Town lattice truss.

6.3.1 - Component stiffness

The overall stiffness of the chord will depend on the stiffnesses of the underlying components. A range of possible stiffness values can be estimated based on earlier analysis and the literature.

6.3.1.1 - Pegged connection stiffness
The range of shear stiffness for pegged connections is based on stiffness estimation from chapter 5 and values from the literature. A range from 40000 to 140000 lb/in encompasses a reasonable variation in peg and member properties and contains all values seen in the only known experimental testing on appropriately sized wooden pegged connections (McFarland-Johnson 1995).

6.3.1.2 - Member stiffness
The range of axial stiffness values for the members is based solely on variation in modulus of elasticity. A consistent cross-section of 3" by 12" and a 48" joint spacing are assumed. Modulus of elasticity is allowed to vary from 1000 to 2000 ksi, which was selected as a relatively inclusive range based on values from the Wood Handbook (Forest Products Laboratory 1999). The stiffness is calculated from these properties as:

$$k_m = \frac{E_m \cdot t_m \cdot d_m}{s}$$

where k_m is the member stiffness, E_m is the modulus of elasticity of the member material, t_m and d_m are the dimensions of the chord members, and s is joint spacing.

6.3.2 - Procedure for determination of stiffness

The apparent axial stiffness at any given location in the chord will be that of two, three, or four chord members, depending on the proximity to chord terminations. However, this assumes a rigid connection between members, ignoring the effect of the pegged connections on the stiffness of the chord. Since force must travel through pegged connections to travel axially along the chord, the shear stiffness of the pegs must be included. Since this shear stiffness tends to be significantly lower than the axial stiffness of the chord members, it is expected to have the potential to significantly reduce the effective stiffness of the tension chords.

To account for the effect of the pegged connections, a longer segment of the chord must be considered. The length of the overall chord, the starting point of the pattern, and the stiffnesses of the members and connections can all be varied for any given chord termination pattern. The objective herein is to develop a predictive relationship between the member and connection stiffnesses and the resulting overall average chord stiffness for all patterns. To do this, a reasonable number of data points must be found for average chord stiffness based on various combinations of member and connection stiffness.

For a given pattern, each combination of member and connection stiffness should yield a single overall effective stiffness value. This value is found as the average stiffness of all of the stiffnesses found over a range of lengths and with all possible starting points for each length. For each pattern length, an integer number of patterns was selected to yield a maximum length that was considered reasonable for the Town lattice truss. Table 6.4 gives the maximum number of patterns and the resulting maximum length for different pattern lengths. Overall lengths are based on an assumed joint spacing of 4 ft.

Table 6.4 - Maximum number of patterns and resulting maximum length for stiffness analysis of chords

Pattern Length (units)	Max. # patterns	Max. Length (ft)
4	12	192
5	9	180
6	8	192
7	7	196
8	6	192

Stiffness values are determined using a simple matrix analysis. Chord members are treated as axial springs and pegged connections are treated as shear springs. A global stiffness matrix is derived based on the connectivity of all members, which will be affected by the pattern being analyzed. Restraints are added to all nodes at one end of the chord structure and a unit displacement is applied to all nodes at the other end of the chord structure. The resulting total axial force at any given cross-section is then equal to the effective stiffness of the chord structure. In this work, the reaction forces at the restrained end are taken to represent the axial force, and hence the axial stiffness.

The set of resulting axial stiffnesses for a given pattern and given combination of member and connection stiffness can have significant variance, especially at shorter lengths. Four example plots of resulting stiffness factor values are shown in Figure 6.14. Stiffness factor represents the fraction of axial stiffness seen compared with the axial stiffness of a solid cross-section, defined by the equation

$$f_s = \frac{K \cdot N}{4 \cdot k_m}$$

where f_s is the stiffness factor, K is the chord stiffness, N is the length of the chord structure in joint spacings, and k_m is the member stiffness.

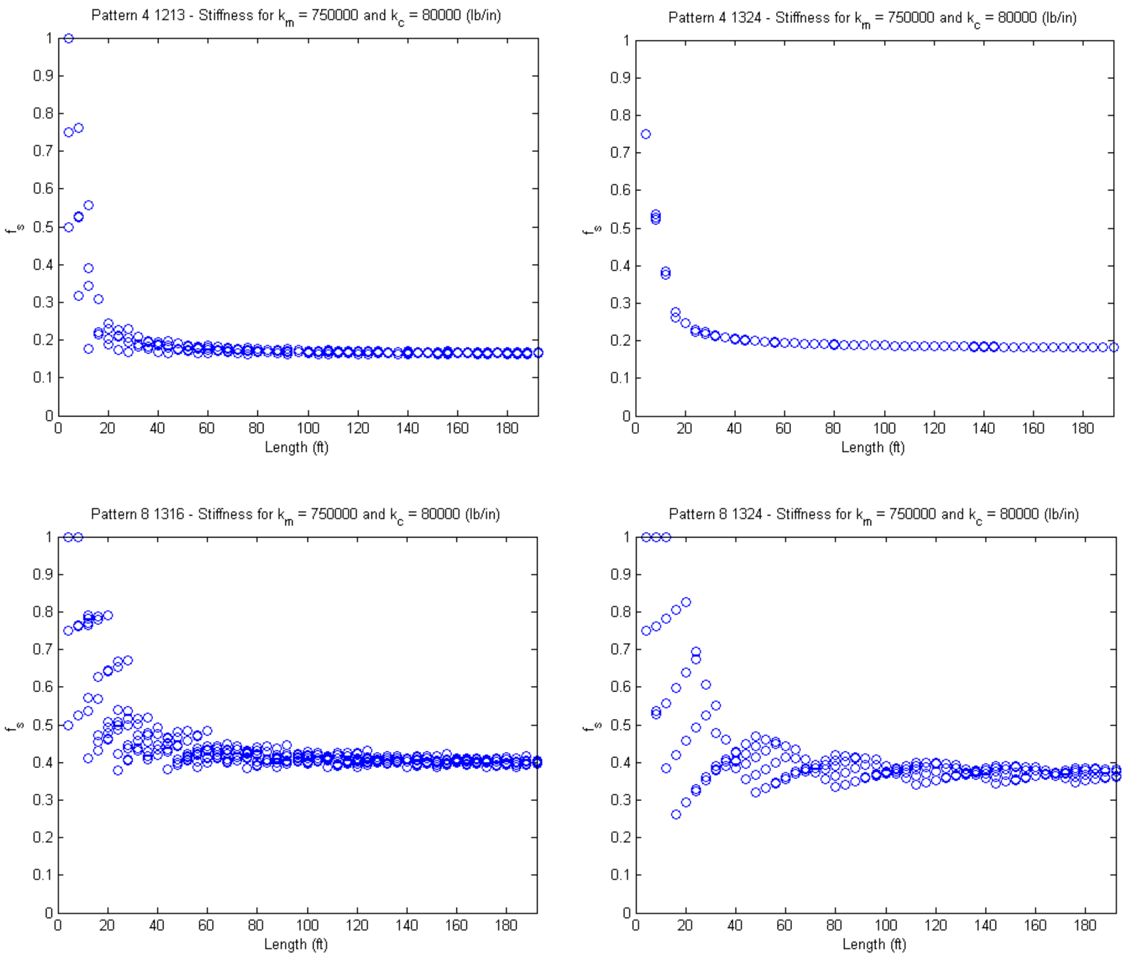


Figure 6.14 - Plots of stiffness factor as a function of length for four patterns

The four plots included in Figure 6.14 show a variety of behaviour. All patterns have significantly higher stiffness values at shorter lengths. At these shorter lengths, the full pattern may not be mobilized and members may even extend the full length of the chord with no terminations. As the chord length increases, values tend to resolve around a consistent mean, although significant variation can still be seen as the pattern starting point is varied. As the length increases, these variations have a decreasing impact on the overall behaviour.

Patterns of the same length may have more or less variation with changing starting point, as can be seen in the difference between patterns of the same length. Pattern 4-1213 shows more variation than 4-1324, which follows an almost perfectly consistent trajectory. Pattern 8-1316 has a distribution that is similar in look to that of pattern 4-1213, although with more variance. Pattern 8-1324 has an even larger variance, although the pattern seems to follow a consistent undulating pattern that is periodic with length.

Finally, it can be noted that the 8-unit patterns offer a significantly higher stiffness than the 4-unit patterns. This is based on the lower frequency of chord terminations, resulting in less force transfer through pegged connections.

Since shorter lengths have significant variance and are not reasonable lengths for use in a Town lattice truss, they will not be included in the determination of mean values. Lengths ranging from about 80 to about 200 feet are used. Exact values vary somewhat by pattern, as an integer number of patterns are used to determine the length; 6 to 12 patterns for 4-unit length, 4 to 9 for 5-unit, 4 to 8 for 6-unit, 3 to 7 for 7-unit, and 3 to 6 for 8-unit. Figure 6.15 contains two example plots showing the reduced data range and a line for the resulting mean value.

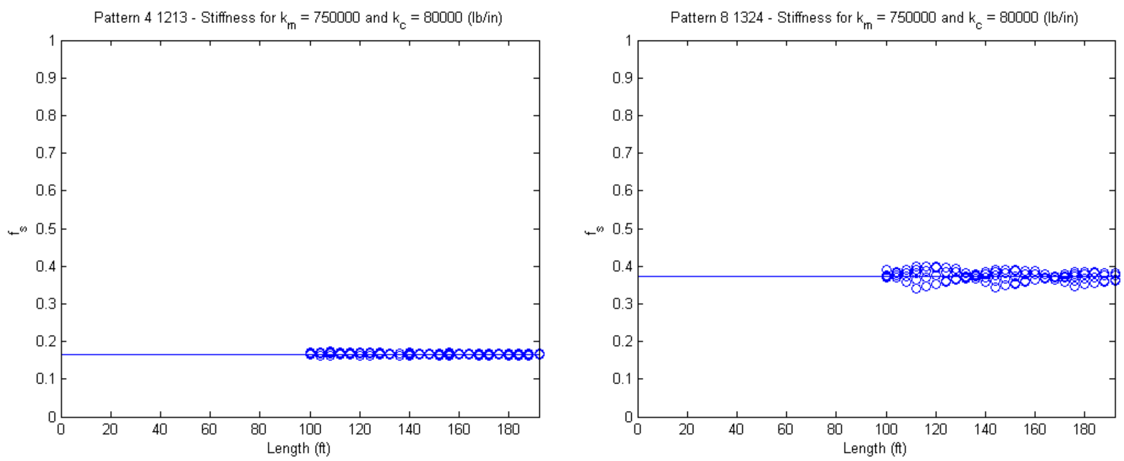


Figure 6.15 - Plots of stiffness as a function of length over a reduced length and including mean

Having derived a set of stiffness factors for a given pattern for a variety of combinations of member and connection stiffnesses, the next step is to determine an appropriate relationship. A non-linear least squares regression will be used to determine the best-fit curve based on the assumed relationship:

$$\tilde{f}_s = C_1 \cdot (k_m)^{C_2} \cdot (k_c)^{C_3}$$

where k_c is the connection stiffness, and C_1, C_2, and C_3 are unknown coefficients.

To perform a least squares regression on the assumed power relationship, the logarithm of both the set of data points and the best-fit equation must be taken. This yields a relationship of:

$$\log f_{s,i} = \log \tilde{f}_{s,i} - \varepsilon_i$$

$$\log f_{s,i} = \log C_1 + C_2 \cdot \log k_{m,i} + C_3 \log k_{c,i} - \varepsilon_i$$

Rearranging this into matrix form for all data points yields:

$$\mathbf{Y} = \mathbf{X} \cdot \mathbf{C} - \boldsymbol{\varepsilon}$$

where

$$\mathbf{Y} = \begin{bmatrix} \log f_{s,1} \\ \log f_{s,2} \\ \vdots \\ \log f_{s,n} \end{bmatrix}, \ \mathbf{X} = \begin{bmatrix} 1 & \log k_{m,1} & \log k_{c,1} \\ 1 & \log k_{m,2} & \log k_{c,2} \\ \vdots & \vdots & \vdots \\ 1 & \log k_{m,n} & \log k_{c,n} \end{bmatrix}, \ \mathbf{C} = \begin{bmatrix} \log C_1 \\ C_2 \\ C_3 \end{bmatrix}, \text{ and } \boldsymbol{\varepsilon} = \begin{bmatrix} \varepsilon_1 \\ \varepsilon_2 \\ \vdots \\ \varepsilon_n \end{bmatrix}$$

The error vector will then have the form

$$\boldsymbol{\varepsilon} = \mathbf{X} \cdot \mathbf{C} - \mathbf{Y}$$

The error measure is defined as the norm of $\boldsymbol{\varepsilon}$.

$$J = \frac{1}{2}\boldsymbol{\varepsilon}^T\boldsymbol{\varepsilon}$$

$$J = \frac{1}{2}\left(\left(\mathbf{C}^T\mathbf{X}^T - \mathbf{Y}^T\right)\cdot\left(\mathbf{XC} - \mathbf{Y}\right)\right)$$

$$J = \frac{1}{2}\left(\mathbf{C}^T\mathbf{X}^T\mathbf{XC} - \mathbf{Y}^T\mathbf{XC} - \mathbf{YC}^T\mathbf{X}^T + \mathbf{Y}^T\mathbf{Y}\right)$$

$$J = \frac{1}{2}\left(\mathbf{C}^T\mathbf{X}^T\mathbf{XC} - 2\cdot\mathbf{YC}^T\mathbf{X}^T + \mathbf{Y}^T\mathbf{Y}\right)$$

The objective of the regression is to minimize the error, which will occur when the derivative of the error measure is equal to zero. The derivative can first be found as:

$$dJ = \frac{1}{2}\left(d\mathbf{C}^T\mathbf{X}^T\mathbf{XC} + \mathbf{C}^T\mathbf{X}^T\mathbf{X}d\mathbf{C} - 2\cdot d\mathbf{C}^T\mathbf{X}^T\mathbf{Y}\right)$$

$$dJ = d\mathbf{C}^T\left(\mathbf{X}^T\mathbf{XC} - \mathbf{X}^T\mathbf{Y}\right)$$

Setting $dJ = 0$ yields:

$$\mathbf{X}^T\mathbf{XC} = \mathbf{X}^T\mathbf{Y}$$

which allows for the determination of the coefficient vector **C** as:

$$\mathbf{C} = \left[\mathbf{X}^T\mathbf{X}\right]^{-1}\mathbf{X}^T\mathbf{Y}$$

Final coefficients can be extracted from the coefficient vector.

Having determined the regression curve, goodness of fit can be determined through the calculation of the coefficient of determination R^2 as:

$$R^2 = 1 - \frac{\sum_{i=1}^{n}\left(f_{s,i} - \tilde{f}_{s,i}\right)^2}{\sum_{i=1}^{n}\left(f_{s,i} - f_{s,m}\right)^2}$$

where $f_{s,m}$ is the mean value of all data points.

An example plot showing data points and the prediction curve for pattern 4-1213 is shown in Figure 6.16. The coefficient of determination for the example pattern is 0.9966.

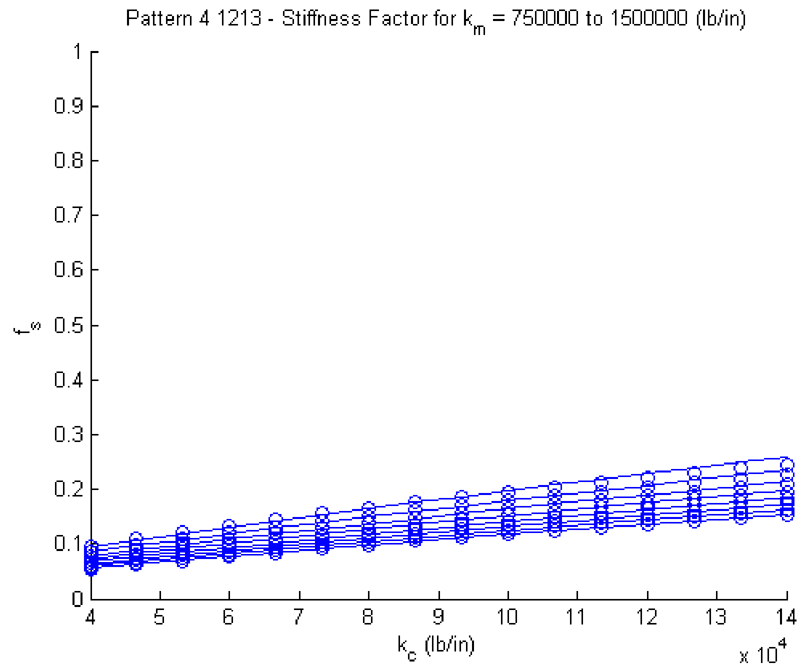


Figure 6.16 - Plot of stiffness factor data points and non-linear regression curve for example pattern

The regression curve appears to offer a reasonable approximation, as would be suggested by the high coefficient of determination, although there is some discrepancy at the extremes of the range. Considering the variability of components and materials, this level of accuracy is considered acceptable for a first approximation of stiffness, which is the primary objective of this component of the research. Complete results for stiffness coefficients are included in Appendix E.

6.3.3 - Comparison between results

It is not generally possible to compare between the stiffness results for different patterns. Only if all three coefficients of one pattern are greater than the respective coefficients of another pattern can the first pattern be definitively stated as yielding higher predicted stiffness for all values. Otherwise, the resulting stiffnesses will depend on the member and connection stiffness values, and numerical values will be needed to allow for comparison. Complete stiffness results for 4-unit patterns, roughly sorted by strength, are given in Table 6.5.

Table 6.5 - 4-unit length patterns with stiffness results, approximately sorted by strength

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c	C_1	C_2	C_3	R^2
4	1 3 1 3	0 6	1 2	2 0		0.8052	-0.7610	0.7818	0.9957
4	1 3 2 4	0 5	1 3	2 1	3 0	0.8175	-0.7716	0.7917	0.9961
4	1 2 4 2	0 5	1 1	2 0		0.7815	-0.7795	0.7989	0.9964
4	1 2 4 3	0 4	1 2	2 1	3 0	0.7654	-0.8031	0.8209	0.9971
4	1 2 1 3	0 4	1 2	2 0		0.7760	-0.7873	0.8062	0.9966
4	1 2 3 1	0 4	1 2	2 0		0.7900	-0.7909	0.8096	0.9967
4	1 2 4 1	0 4	1 2	2 0		0.7616	-0.8026	0.8205	0.9970
4	1 3 3 1	0 4	1 2	2 0		0.7869	-0.7899	0.8089	0.9966
4	1 2 3 4	0 3	1 2	2 1	3 0	0.7699	-0.8147	0.8319	0.9973
4	1 2 1 2	0 3	1 1	2 0		0.7315	-0.8060	0.8238	0.9971
4	1 2 1 4	0 3	1 1	2 0		0.7506	-0.8102	0.8277	0.9972
4	1 2 2 4	0 3	1 1	2 0		0.7455	-0.8075	0.8252	0.9971
4	1 2 2 1	0 2	1 1	2 0		0.7017	-0.8305	0.8466	0.9977
4	1 2 2 3	0 2	1 1	2 0		0.7052	-0.8301	0.8461	0.9977

It may be possible to compare between stiffness extremes to determine if the resulting stiffnesses are consistently higher or lower between two patterns. For the analysis, member stiffnesses ranged from 750000 to 1500000 lb/in and connection stiffnesses ranged from 40000 to 140000 lb/in. These were assumed to be reasonably large ranges that would easily encompass the functional range of stiffnesses likely to be seen in Town lattice truss bridges. Taking an envelope defined by a smaller range is reasonable for the purposes of comparison. Values at 1/4 and 3/4 of the given ranges will be used as upper and lower bounds to estimate a common stiffness envelope. This results in connection stiffness ranging from a k_{cl} of 65000 lb/in to a k_{ch} of 115000 lb/in and member stiffness ranging from a k_{ml} of 937500 lb/in to a k_{mh} of 1312500 lb/in. The results for stiffness factor from these values for 4-unit patterns are shown in Table 6.6.

Table 6.6 - 4-unit patterns, approximately sorted by strength, showing envelope stiffnesses

					Stiffness Factor, f_s			
N	Pattern	C_1	C_2	C_3	k_{ml},k_{cl}	k_{mh},k_{cl}	k_{ml},k_{ch}	k_{mh},k_{ch}
4	1 3 1 3	0.8052	-0.7610	0.7818	0.1329	0.1029	0.2077	0.1607
4	1 3 2 4	0.8175	-0.7716	0.7917	0.1302	0.1004	0.2046	0.1578
4	1 2 4 2	0.7815	-0.7795	0.7989	0.1210	0.0931	0.1909	0.1469
4	1 2 4 3	0.7654	-0.8031	0.8209	0.1093	0.0834	0.1746	0.1333
4	1 2 1 3	0.7760	-0.7873	0.8062	0.1171	0.0898	0.1855	0.1423
4	1 2 3 1	0.7900	-0.7909	0.8096	0.1178	0.0903	0.1870	0.1433
4	1 2 4 1	0.7616	-0.8026	0.8205	0.1091	0.0832	0.1742	0.1329
4	1 3 3 1	0.7869	-0.7899	0.8089	0.1181	0.0905	0.1874	0.1436
4	1 2 3 4	0.7699	-0.8147	0.8319	0.1058	0.0804	0.1701	0.1293
4	1 2 1 2	0.7315	-0.8060	0.8238	0.1037	0.0790	0.1659	0.1265
4	1 2 1 4	0.7506	-0.8102	0.8277	0.1048	0.0798	0.1681	0.1280
4	1 2 2 4	0.7455	-0.8075	0.8252	0.1051	0.0801	0.1683	0.1282
4	1 2 2 1	0.7017	-0.8305	0.8466	0.0915	0.0692	0.1482	0.1121
4	1 2 2 3	0.7052	-0.8301	0.8461	0.0919	0.0695	0.1489	0.1126

It can be seen that higher stiffness is well correlated with higher strength. This makes sense, since the higher strength patterns tend to have failure modes that include the most connections. Since connection stiffness is much smaller than member stiffness, it is along these connection-only failure planes where it is expected to see the greatest deformation. Increasing the number of connections that must deform will increase the overall stiffness of the chord structure.

6.3.4 - Results and Conclusions

Stiffness coefficients and envelope stiffness factors for the set of highest strength patterns, defined earlier, are given in Table 6.7. It can be seen that the average chord stiffnesses of the patterns increase significantly as the member length is increased. It can also be seen that differences between stiffness factors in the highest strength patterns of each member length are relatively small. Therefore, stiffness cannot be used as a determining factor in selecting a pattern within a given member length. The increase of chord stiffness with increasing member length does imply an additional motivation for increasing member length when possible.

Table 6.7 - Stiffness coefficients and envelope stiffness factors for highest strength patterns

N	Pattern				C_1	C_2	C_3	R^2	k_{ml},k_{cl}	k_{mh},k_{cl}	k_{ml},k_{ch}	k_{mh},k_{ch}
4	1	3	1	3	0.8052	-0.7610	0.7818	0.9957	0.1329	0.1029	0.2077	0.1607
4	1	3	2	4	0.8175	-0.7716	0.7917	0.9961	0.1302	0.1004	0.2046	0.1578
5	1	3	5	2	0.8992	-0.6893	0.7131	0.9935	0.1858	0.1473	0.2790	0.2213
6	1	4	1	4	0.9180	-0.6028	0.6288	0.9903	0.2451	0.2001	0.3509	0.2865
6	1	4	2	5	0.9342	-0.6094	0.6352	0.9906	0.2444	0.1991	0.3511	0.2861
7	1	4	7	3	0.9388	-0.5387	0.5647	0.9881	0.2974	0.2481	0.4104	0.3424
7	1	4	6	2	0.9545	-0.5476	0.5737	0.9884	0.2956	0.2459	0.4101	0.3411
8	1	5	1	5	0.9228	-0.4712	0.4967	0.9854	0.3479	0.2969	0.4619	0.3942
8	1	5	3	7	0.9510	-0.4845	0.5100	0.9861	0.3462	0.2941	0.4631	0.3934

6.4 - Summary

A comprehensive assessment of chord termination patterns was conducted. All reasonable patterns were identified for member lengths from 4 to 8 units long, and these patterns were compared based on strength. Potential strongest patterns were identified for each member length, and the strength of these patterns can be evaluated based on a simple combination of the strengths of the underlying properties.

Stiffness coefficients were also developed for all patterns and presented for the strongest patterns. Stiffness was found to correlate with strength and therefore cannot be used as a method to select between patterns of similar strength. The final stiffness equation yields a stiffness factor that represents the ratio of stiffness of a tension chord with a given pattern to a compression, for which the pattern will have no effect on stiffness.

Strength and stiffness properties for chords will be used in the following chapter to determine the moment capacity of Town lattice trusses. A final methodology for the design of Town lattice trusses will be established based on the use of these properties.

6.5 - References

ASTM (2000). D 5652 - 95 Standard Test Methods for Bolted Connections on Wood and Wood-Based Products. American Society for Testing and Materials, West Conshohocken, PA.

Forest Products Laboratory (1999). Wood handbook - Wood as an engineering material. Gen. Tech. Rep. FPL-GTR-113. Department of Agriculture, Forest Service, Forest Products Laboratory: 463p.

McFarland-Johnson (1995). Wooden peg tests : their behavior and capacity as used in Town lattice trusses. Vermont Department of Transportation under Contract TH 9290 - Long Term Covered Bridge Study.

Chapter 7 – The Town Lattice Truss: Design

Having developed rules for determining component strengths and stiffnesses, it is now necessary to determine the overall capacity of the Town lattice truss. Connection and member strengths and stiffnesses determine the chord strength and stiffness based on rules developed in Chapter 6. The chord strength and stiffness, in turn, determine the moment capacity of the truss. This final moment capacity of the truss will determine the allowable span of a given cross-section when subjected to its appropriate loading.

Moment capacity is considered the first, and most important, design parameter for the Town lattice truss. Once rules are developed for the determination of moment capacity, initial designs and geometries can be established. After this point, there are a multitude of other design details that must be assessed before such a bridge is ready to be constructed, but these types of details will be most effectively explored through the process of design and construction of a prototype bridge, which is beyond the scope of this work.

In this chapter, the relationship between chord strength and stiffness and truss moment capacity is first developed. Assuming the bottommost chord fails in tension and that forces are distributed to the chords proportional to their effective stiffness allows for a simple calculation of moment capacity for any truss geometry. The specific details of the calculation of moment capacity are described through the analysis of a typical Town lattice truss. Results from this analysis are used to recommend general parameters to be used in the design of Town lattice truss bridges.

Next, a simple design methodology is proposed. The methodology incorporates all equations and information that must be used in the development of an adequate design for a given span. The methodology incorporates an iterative variation of truss height to allow for the determination of an efficient geometry.

Finally, the design methodology is applied for a range of spans with different maximum member lengths. The resulting designs are compared with other bridge timber bridge systems to assess the comparative efficiency of the Town lattice truss.

7.1 - Relationship between maximum chord capacity and maximum moment capacity

With the assumption that all moment is carried by the chords, the Town lattice truss will have up to four axial elements that contribute to the moment capacity. The number will be less if the lower top chord or the upper bottom chord are excluded from the truss, as is seen in some existing bridges. For the purposes of this work, four lines of chords will be assumed in all trusses.

The bottommost tension chord will be assumed to reach the maximum chord capacity at failure. This chord capacity can be determined based on the component strengths and the failure modes presented in Chapter 6. Each of the other chords will have an unknown axial force which must be determined to evaluate the overall moment capacity. The moment capacity of the truss can be calculated as the sum of the moments exerted by each of the of the chord forces around the neutral axis of the section. This is illustrated in Figure 7.1 and defined by the equation:

$$M = F_{LB} \cdot (H - c) + F_{UB} \cdot (H - h - c) + F_{LT} \cdot (c - h) + F_{UT} \cdot (c)$$

where H is the truss height, defined as the center-to-center distance between the topmost and bottommost chords, h is the height between rows of joints, c is the depth of the neutral axis, and F_{LB}, F_{UB}, F_{LT}, and F_{UT} are the force in the lower-bottom, upper-bottom, lower-top, and upper-top chords, respectively.

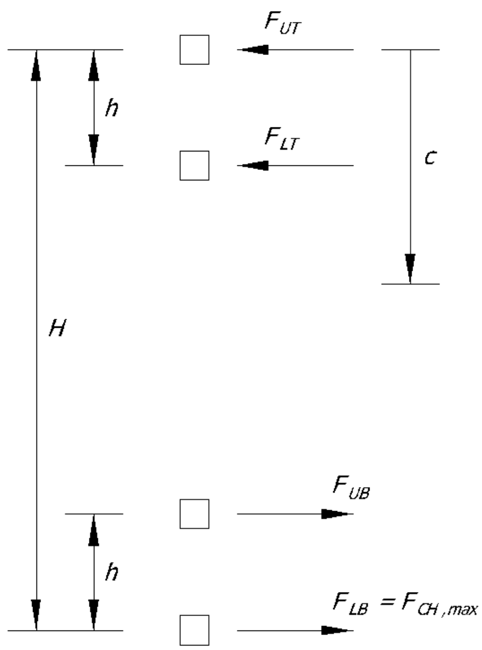


Figure 7.1 - Geometry and forces of four chords in Town lattice truss under bending moment

One relationship can be established for the chord forces using horizontal equilibrium. This yields the equation:

$$F_{LT} + F_{UT} = F_{LB} + F_{UB}$$

This only provides one equation, which is not enough to solve for the four unknown variables, three chord forces and the depth of the neutral axis. To be able to determine all of the unknowns, some assumptions must be made.

It is first assumed that the Town lattice truss will behave as a beam and plane sections will remain plane. This provides a relationship between the displacements seen in each of the chords. In order to convert these displacements into axial forces, a stiffness value is needed for each of the chords.

The top chords, assumed to act in compression, will have the full axial stiffness of the four members. At the time of failure, all chord termination gaps in the compression chords will have closed and the pegged connections will no longer have an impact. The bottom chords, assumed to act in tension, will have a reduced stiffness due to the low shear stiffness of the connections and based on the chord termination pattern.

A linear elastic stiffness will be assumed for all components. This is considered a reasonable assumption since the bottom chord failure may involve a tension failure, which is generally brittle and does not exhibit large inelastic deformations. The resulting section is shown in Figure 7.2 with the deformed plane section shown and stiffnesses labeled. Factor α represents the reduction in stiffness of the tension chords and correlates with the stiffness factor, f_s, defined in Chapter 6.

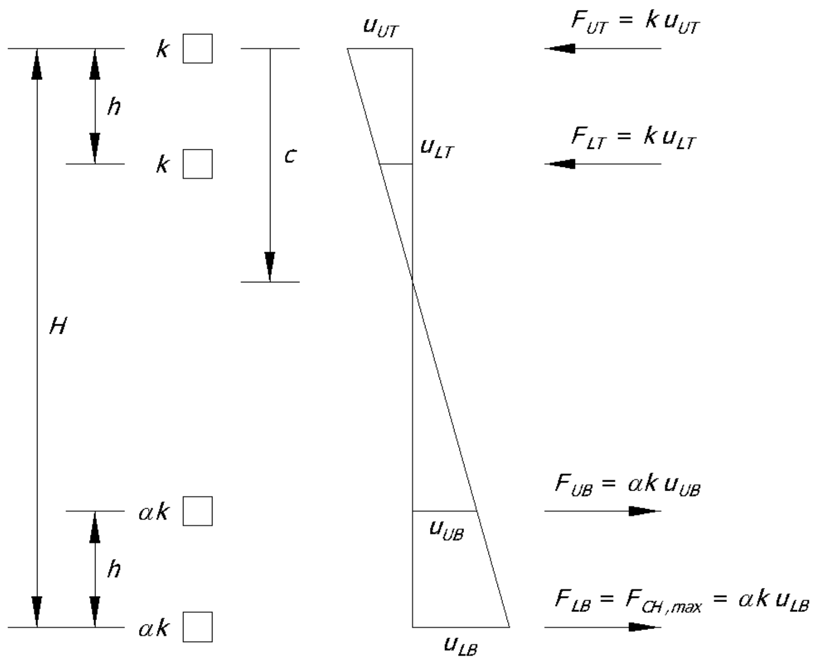


Figure 7.2 - Plane section and resulting forces in Town lattice truss

Based on the similar triangles shown in Figure 7.2, the displacement of each chord can be found as a function of the displacement of the bottom chord, u_{LB}.

$$u_{UB} = \left(\frac{H-c-h}{H-c}\right)\cdot u_{LB}$$

$$u_{LT} = \left(\frac{c-h}{H-c}\right)\cdot u_{LB}$$

$$u_{UT} = \left(\frac{c}{H-c}\right)\cdot u_{LB}$$

Knowing that

$$u_{LB} = \frac{F_{CH,\max}}{\alpha k}$$

each of the unknown chord forces can be found as a function of $F_{CH,max}$.

$$F_{UB} = \alpha k \cdot u_{UB} = \alpha k \cdot \left(\frac{H-c-h}{H-c}\right) \cdot u_{LB} = \left(\frac{H-c-h}{H-c}\right) \cdot F_{CH,\max}$$
$$F_{LT} = k \cdot u_{LT} = k \cdot \left(\frac{H-c-h}{H-c}\right) \cdot u_{LB} = \frac{1}{\alpha}\left(\frac{c-h}{H-c}\right) \cdot F_{CH,\max}$$
$$F_{UT} = k \cdot u_{UT} = k \cdot \left(\frac{c}{H-c}\right) \cdot u_{LB} = \frac{1}{\alpha}\left(\frac{c}{H-c}\right) \cdot F_{CH,\max}$$

The depth of the neutral axis, c, is still unknown. This can now be determined using the equation of horizontal equilibrium.

$$F_{LT} + F_{UT} = F_{LB} + F_{UB}$$
$$\frac{1}{\alpha}\left(\frac{c-h}{H-c}\right) \cdot F_{CH,\max} + \frac{1}{\alpha}\left(\frac{c}{H-c}\right) \cdot F_{CH,\max} = \left(\frac{H-c-h}{H-c}\right) \cdot F_{CH,\max} + F_{CH,\max}$$
$$(c-h) + (c) = \alpha(H-c-h) + \alpha(H-c)$$
$$c + c + \alpha c + \alpha c = \alpha H - \alpha h + \alpha H + h$$
$$c = \frac{h - \alpha h + 2\alpha H}{2 + 2\alpha}$$

Having solved for the location of the neutral axis, all chord forces at failure are known, and the moment capacity of the section can be calculated.

7.2 - Example capacity analysis

An example analysis of a typical Town lattice truss cross-section will now be conducted to illustrate the procedure described above.

7.2.1 - Truss properties

A variety of geometric properties are selected based on average properties recommended in Chapter 4. Initially selected parameters include joint spacing, web and chord member dimensions, web angle, and the number of lines of joints. This general layout and associated properties are given in Figure 7.3.

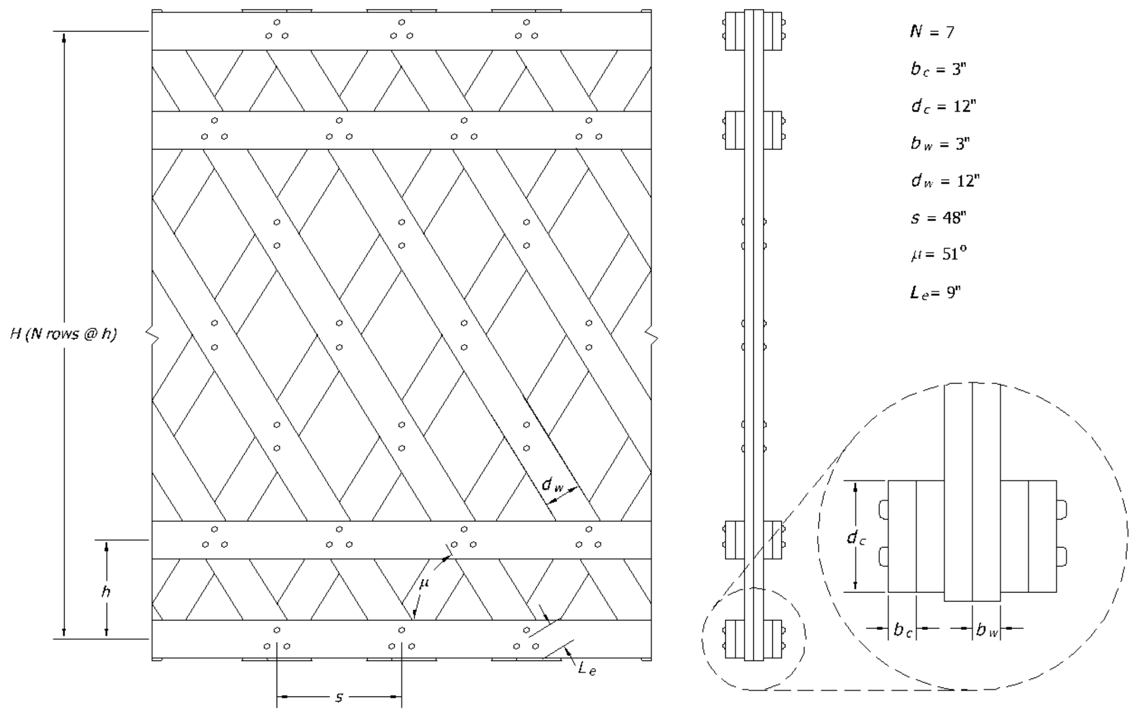


Figure 7.3 - Typical Town lattice truss layout and properties

Based on the on the selected properties, height values h and H can be calculated.

$$h = \frac{s}{2} \cdot \tan \mu = \frac{48}{2} \cdot \tan 51^\circ = 29.6 \text{ ''}$$
$$H = (N-1) \cdot h = 6 \cdot 29.6 = 177.8 \text{ ''}$$

Several general connection properties must also be selected. 2" diameter pegs will be used throughout and chord connections will use 3 or 4 pegs. Rotational properties of pegged connections will be ignored for the purposes of this analysis.

Finally, material properties must be selected. Wood for both the member material and the pegs will be assumed to be at the upper end of Group B woods, as defined in Table 3.6. Therefore, strength properties will be those for Group B woods, e.g. permissible tension stress F_t = 0.73 ksi, while specific gravity and dead load will be based on the dividing value between Group B and Group A, i.e. G_{12} = 0.65 and γ = 50 lb/ft^3.

7.2.2 - Component properties

7.2.2.1 - Chord member properties

Member capacity is calculated simply as the permissible tension stress multiplied by a reduced cross-sectional area at the location of the pegged connections. With either 3- or 4-peg connections, the worst case will see 2 peg holes at the same cross-section yielding an effective cross-sectional area of:

$$A_t = b_c \cdot (d_c - 2 \cdot D) = 3 \cdot (12 - 2 \cdot 2) = 24 \text{ in}^2$$

This yields a member design strength of:

$$F_{m,\max} = F_t \cdot A_t = 0.73 \cdot 10^3 \cdot 24 = 17520 \text{ lbs}$$

The calculation of member stiffness requires modulus of elasticity for the member material. The relationship for the modulus of elasticity for hardwoods from the Wood Handbook (Forest Products Laboratory 1999) can be used.

$$E = 2.39 \cdot {G_{12}}^{0.7} \cdot 10^6 = 2.39 \cdot 0.65^{0.7} \cdot 10^6 = 1.768 \cdot 10^6 \text{ psi}$$

Chord member stiffness can be calculate based on the modulus of elasticity, the full cross-sectional area of a chord member, and the joint spacing.

$$k_m = \frac{E \cdot A}{s} = \frac{1.768 \cdot 10^6 \cdot 3 \cdot 12}{48} = 1.326 \cdot 10^6 \text{ lb/in}$$

7.2.2.2 - Pegged connection properties

The equations to be used to determine the design strength of a pegged connection in a Town lattice truss are:

Mode $\mathbf{I_d}$ $\quad ZI_d = 662.5 \cdot D \cdot t \cdot G_{dd}^{2.04}$ or $ZI_d = \dfrac{D \cdot t \cdot F_{ed}}{8}$

Mode $\mathbf{V_d}$ $\quad ZV_d = 538.8 \cdot D^2 \cdot G_{dd}^{0.84}$ or $ZV_d = 0.223 \cdot D^2 \cdot F_{ev}$

In this case, strength properties are not available and estimation will be done using the specific gravity of Group B wood. The specific given is for 12% moisture content, while the equations above are based on dry specific gravity. The values can be approximately converted using the equation from Wilkinson (1991)

$$G_d \cong 1.067 \cdot G_{12}$$

This produces yield mode strengths for a single peg of:

$$ZI_d = 662.5 \cdot D \cdot t \cdot G_{dd}^{2.04} = 662.5 \cdot 2 \cdot 3 \cdot (1.067 \cdot 0.65)^{2.04} = 1884 \text{ lbs}$$

$$ZV_d = 538.8 \cdot D^2 \cdot G_{dd}^{0.84} = 538.8 \cdot 2^2 \cdot (1.067 \cdot 0.65)^{0.84} = 1584 \text{ lbs}$$

The shear mode ($\mathbf{V_d}$) dominates since the design strength is lower, and the resulting connection strength will be the load for a single peg multiplied by the number of pegs in the connection. Thus, a 3-peg connection will have an allowable load of 4753 lbs and a 4-peg connection will have an allowable load of 6337 lbs.

The stiffness of the pegged connection will be approximated using the formula developed in Chapter 5. For 3" thick members, the connection stiffness was found to be:

$$k_c = N_{peg} \cdot 0.2829 \cdot E^{0.1453} \cdot D^{1.436} \cdot {k_b}^{0.8548}$$

The modulus of elasticity will be the same as that found above for the Group B wood. The only remaining unknown is the bearing stiffness, k_b. An average bearing stiffness value of 20000 psi/in will be used.

$$k_c = 3 \cdot 0.2829 \cdot \left(1.768 \cdot 10^6\right)^{0.1453} \cdot 2^{1.436} \cdot 20000^{0.8548} = 88170 \text{ lb/in for 3-peg}$$
$$k_c = 4 \cdot 0.2829 \cdot \left(1.768 \cdot 10^6\right)^{0.1453} \cdot 2^{1.436} \cdot 20000^{0.8548} = 117560 \text{ lb/in for 4-peg}$$

7.2.3 - Chord properties

Given values for member and connection design strength, it is now possible to assess chord capacity based on the failure modes identified in Chapter 6. For a given member length, there are 1, 2, or 3 patterns which may offer the highest capacity. Each of these patterns has a unique set of failure modes, consisting of a sum of a certain number of members and connections which all must fail simultaneously to cause the chord to fail. The failure mode with the lowest failure load will define the mode of failure and the failure load of the pattern. In selecting a pattern for a given member length, the pattern with the highest failure load will be selected. Results for failure loads for patterns of 4-unit to 8-unit in length are presented in Table 7.1 for connections with 3 pegs and in Table 7.2 for connections with 4 pegs.

Table 7.1 - Chord allowable load results for various member lengths with 3-peg connections

Pattern	Allowable Load (lbs)	Mode Counts and Failure Loads (lbs)							
		F_m	F_c	F_m	F_c	F_m	F_c	F_m	F_c
4 - 1313	27026	0	6	1	2	2	0		
		28519		27026		35040			
4 - 1324	23766	0	5	1	3	2	1	3	0
		23766		31779		39793		52560	
5 - 1352	28519	0	6	1	3	2	1	3	0
		28519		31779		39793		52560	
6 - 1414	31779	0	9	1	3	2	0		
		42778		31779		35040			
6 - 1425	36533	0	8	1	4	2	1	3	0
		38025		36533		39793		52560	
7 - 1473	36533	0	9	1	4	2	1	3	0
		42778		36533		39793		52560	
7 - 1462	38025	0	8	1	5	2	1	3	0
		38025		41286		39793		52560	
8 - 1515	35040	0	12	1	4	2	0		
		57038		36533		35040			
8 - 1526	39793	0	11	1	5	2	1	3	0
		52285		41286		39793		52560	
8 - 1537	44546	0	10	1	6	2	2	3	0
		47532		46039		44546		52560	

Table 7.2 - Chord allowable load results for various member lengths with 4-peg connections

Pattern	Allowable Load (lbs)	Mode Counts and Failure Loads (lbs)							
		F_m	F_c	F_m	F_c	F_m	F_c	F_m	F_c
4 - 1313	30195	0	6	1	2	2	0		
		38025		30195		35040			
4 - 1324	31688	0	5	1	3	2	1	3	0
		31688		36533		41378		52560	
5 - 1352	36533	0	6	1	3	2	1	3	0
		38025		36533		41378		52560	
6 - 1414	35040	0	9	1	3	2	0		
		57038		36533		35040			
6 - 1425	41378	0	8	1	4	2	1	3	0
		50700		42870		41378		52560	
7 - 1473	41378	0	9	1	4	2	1	3	0
		57038		42870		41378		52560	
7 - 1462	41378	0	8	1	5	2	1	3	0
		50700		49208		41378		52560	
8 - 1515	35040	0	12	1	4	2	0		
		76051		42870		35040			
8 - 1526	41378	0	11	1	5	2	1	3	0
		69713		49208		41378		52560	
8 - 1537	47715	0	10	1	6	2	2	3	0
		63375		55545		47715		52560	

The chord stiffness factor can be estimated for each of the recommended patterns. The equation

$$\alpha = C_1 \cdot (k_m)^{C_2} \cdot (k_c)^{C_3}$$

is used, and the coefficients for each pattern can be taken from Table 6.6. Results for the calculation of stiffness factor, α, are given in Table 7.3 for connections with 3 pegs and in Table 7.4 for connections with 4 pegs.

Table 7.3 - Stiffness factor results for various member lengths with 3-peg connections

Pattern	C_1	C_2	C_3	α
4 - 1313	0.8052	-0.7610	0.7818	0.130
4 - 1324	0.8175	-0.7716	0.7917	0.127
5 - 1352	0.8992	-0.6893	0.7131	0.182
6 - 1414	0.9180	-0.6028	0.6288	0.241
6 - 1425	0.9342	-0.6094	0.6352	0.240
7 - 1473	0.9388	-0.5387	0.5647	0.293
7 - 1462	0.9545	-0.5476	0.5737	0.291
8 - 1515	0.9228	-0.4712	0.4967	0.344
8 - 1526	0.9359	-0.4754	0.5007	0.344
8 - 1537	0.9510	-0.4845	0.5100	0.342

Table 7.4 - Stiffness factor results for various member lengths with 4-peg connections

Pattern	C_1	C_2	C_3	α
4 - 1313	0.8052	-0.7610	0.7818	0.162
4 - 1324	0.8175	-0.7716	0.7917	0.159
5 - 1352	0.8992	-0.6893	0.7131	0.223
6 - 1414	0.9180	-0.6028	0.6288	0.289
6 - 1425	0.9342	-0.6094	0.6352	0.288
7 - 1473	0.9388	-0.5387	0.5647	0.345
7 - 1462	0.9545	-0.5476	0.5737	0.343
8 - 1515	0.9228	-0.4712	0.4967	0.397
8 - 1526	0.9359	-0.4754	0.5007	0.398
8 - 1537	0.9510	-0.4845	0.5100	0.396

7.2.4 - Moment capacity

The moment capacity of the truss is determined by first calculating the neutral axis depth based on the stiffness factor, and then using the resulting value to solve for the three unknown chord forces. The moments exerted my all four chord forces are finally summed to determine the final moment capacity. Equations were presented in Section 7.1 and results for moment capacity are given in Table 7.5.

Table 7.5 - Moment capacity for various member lengths with 3- or 4-peg connections

	3-Peg					4-Peg				
Member Length (units)	4	5	6	7	8	4	5	6	7	8
Pattern	1313	1352	1425	1462	1537	1324	1352	1425	1462	1537
$F_{CH,max}$ (lbs)	27026	28519	36533	38025	44546	31688	36533	41378	41378	47715
Chord Stiffness Factor	0.130	0.182	0.240	0.291	0.342	0.159	0.223	0.288	0.343	0.396
Neutral Axis Depth (in)	31.8	37.6	43.5	48.2	52.6	35.2	41.9	48.0	52.7	56.9
F_{ut} (lbs)	45450	42083	49286	48609	54691	49061	50389	53036	50745	56632
F_{lt} (lbs)	3116	8927	15717	18745	23861	7730	14713	20275	22209	27108
F_{ub} (lbs)	21540	22491	28471	29328	34005	25104	28569	31933	31576	36025
Moment Capacity (kip-ft)	658.8	678.3	854.1	879.4	1021.1	760.5	857.8	957.4	948.2	1085.0

7.2.5 - Determination of dead load

The moment capacity must be sufficient to resist both the self-weight of the bridge and the applied live loading from vehicles and pedestrians. The self-weight of the trusses can be calculated based on the geometric properties already defined. In addition to the self-weight of the truss, the deck structure and roof structure will contribute to the dead load.

The Town lattice truss can be divided into a repeating series of fundamental units as shown in Figure 7.4. The volume of material in a fundamental unit can be multiplied by the appropriate unit weight of wood and divided by joint spacing, *s*, which also represents the length of the fundamental unit, to yield a uniformly distributed dead load for each truss.

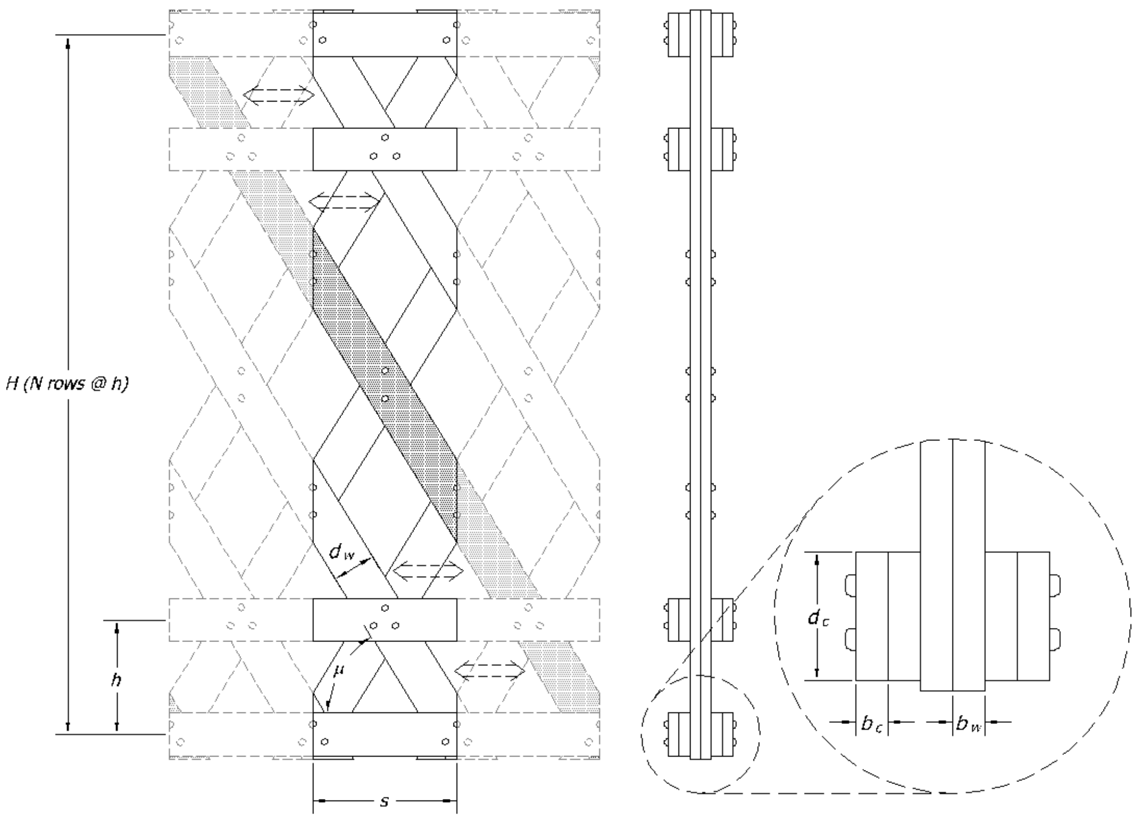


Figure 7.4 - Fundamental repeating unit of Town lattice truss

The volume of the chord members in the fundamental unit can be calculated as:

$$V_{chords} = 4 \cdot b_c \cdot d_c \cdot s \cdot N_c$$

The volume of the chords can also be calculated. As can be seen from the highlighted members in Figure 7.4, each fundamental unit contains the volume of one full-length web member in each layer.

$$V_{web} = 2 \cdot L_w \cdot b_w \cdot d_w \cdot L_w$$

where L_w is the total length of a web member. This length can be calculated based on the height of the truss, H, the angle of the web members, μ, and the extension of the web members beyond the centre lines of the outermost chords, L_e. This is illustrated in Figure 7.5 and yields the equation:

$$L_w = \frac{H}{\sin \mu} + 2 \cdot L_e$$

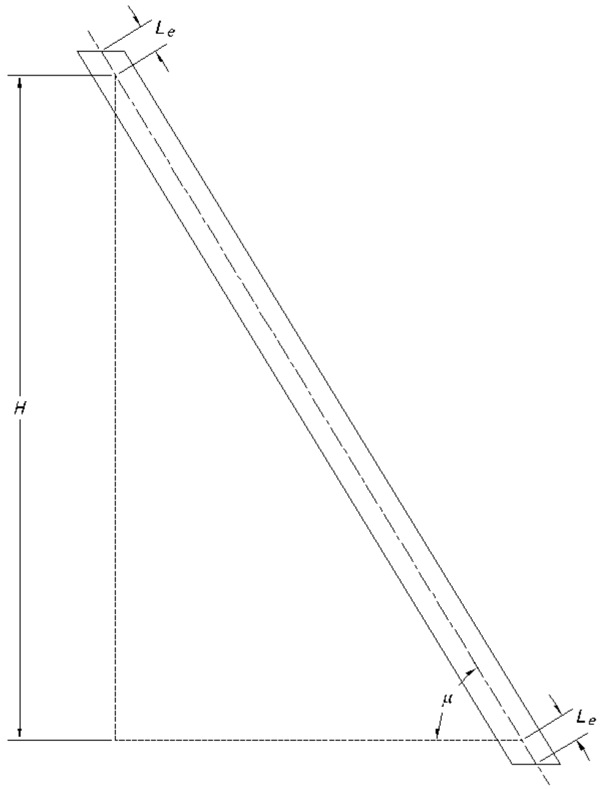


Figure 7.5 - Web member length

Combining the contributions of the chords and the web yields a distributed dead load per truss of:

$$w_{D,truss} = \frac{(V_{chords} + V_{web}) \cdot \gamma}{s} = \frac{\gamma}{s} \cdot \left(4 \cdot b_c \cdot d_c \cdot s \cdot N_c + 2 \cdot b_w \cdot d_w \cdot \left(\frac{H}{\sin \mu} + 2 \cdot L_e \right) \right)$$

$$w_{D,truss} = \frac{50}{48/12} \cdot \left(4 \cdot 3/12 \cdot 12/12 \cdot 48/12 \cdot 4 + 2 \cdot 3/12 \cdot 12/12 \cdot \left(\frac{177.8/12}{\sin 51^\circ} + 2 \cdot 9/12 \right) \right)$$

$$w_{D,truss} = 12.5 \cdot (16 + 0.5 \cdot (19.1 + 1.5))$$

$$w_{D,truss} = 328.6 \text{ lb/ft/truss}$$

The deck will have crossbeams, the weight of which must be divided over their spacing, and a continuous deck. This gives a contribution to dead load of:

$$w_{D,deck} = \left(\frac{1}{k_{dcb} \cdot s} \cdot b_{dcb} \cdot d_{dcb} \cdot L_{dcb} + d_d \cdot w_d \right) \cdot \gamma$$

where b_{dcb}, d_{dcb}, and L_{dcb} are the dimensions of the crossbeams, k_{dcb} is a factor that relates crossbeam spacing to joint spacing, d_d is the depth of the deck, and w_d is the width of the deck.

The roof will have crossbeams, cross-bracing, diagonals, rafters, sheathing, and roofing material. Crossbeams can be determined and will need to be divided over their spacing.

The other elements are all highly variable and will be estimated by a single continuous roofing layer over the roof area. This yields a contribution to dead load of:

$$w_{D,roof} = \left(\frac{1}{k_{rcb} \cdot s} \cdot b_{rcb} \cdot d_{rcb} \cdot L_{rcb} + t_r \cdot \frac{L_{rcb}}{\cos\theta} \right) \cdot \gamma$$

where b_{rcb}, d_{rcb}, and L_{rcb} are the dimensions of the crossbeams, k_{rcb} is a factor that relates crossbeam spacing to joint spacing, t_r is the equivalent thickness of the roof, and θ is the angle of the roof.

Values to be used in the estimation of deck and roof dead load are given in Table 7.6

Table 7.6 - Values used in the calculation of estimated dead load

Deck		Roof	
b_{dcb}	13.5”	b_{rcb}	10”
d_{dcb}	7”	d_{rcb}	6”
L_{dcb}	17’	L_{rcb}	17’
k_{dcb}	0.5	k_{rcb}	3
d_d	4”	t_r	3”
w_d	15’	θ	32°

The contribution to dead load of the deck can be estimated as:

$$w_{D,deck} = \left(\frac{1}{k_{dcb} \cdot s} \cdot b_{dcb} \cdot d_{dcb} \cdot L_{dcb} + d_d \cdot w_d \right) \cdot \gamma$$

$$w_{D,deck} = \left(\frac{1}{0.5 \cdot 48/12} \cdot 13.5/12 \cdot 7/12 \cdot 17 + 4/12 \cdot 15 \right) \cdot 50$$

$$w_{D,deck} = 528.9 \text{ lb/ft}$$

The contribution to dead load of the roof can be estimated as:

$$w_{D,roof} = \left(\frac{1}{k_{rcb} \cdot s} \cdot b_{rcb} \cdot d_{rcb} \cdot L_{rcb} + t_r \cdot \frac{L_{rcb}}{\cos\theta} \right) \cdot \gamma$$

$$w_{D,roof} = \left(\frac{1}{3 \cdot 48/12} \cdot 10/12 \cdot 6/12 \cdot 17 + 3/12 \cdot \frac{17}{\cos 32^\circ} \right) \cdot 50$$

$$w_{D,roof} = 281.1 \text{ lb/ft}$$

These equivalent distributed loads will all contribute to a bending moment that must be carried by the trusses of the bridge, however this value will depend on the span of the bridge. Determining the maximum span of the bridge is the ultimate goal of the analysis and will be addressed in the following section.

7.2.6 - Live load capacity and allowable span

The final step in the analysis is to determine the maximum possible span of a bridge supported by the given truss. This must be done through an iterative process, since both the dead and live loads will be dependent on the span. The procedure used herein is one where the live load moment capacity for a given span is evaluated and compared with the required live load moment capacity.

Live load capacity is defined herein as the capacity of the bridge to support moment beyond that applied by the bridge's self weight. Since live loading is of a short duration, the moment capacity of the bridge can be increased by a load duration factor, C_d, as discussed in Chapter 3. A load duration factor of 1.25 is recommended. This yields an equation for live load capacity of:

$$M_{\max,LL} = 2 \cdot C_d \cdot M_{\max,truss} - \frac{w_{D,tot} \cdot L^2}{8}$$

which must satisfy the requirement:

$$M_{\max,LL} > M_{LL}$$

The maximum allowable span of the truss is the maximum span that satisfies the requirement that live load capacity exceed applied live load moment. Final results for maximum allowable span for the typical truss are given in Table 7.7.

Table 7.7 – Maximum span for various member lengths with 3- or 4-peg connections

	3-Peg					4-Peg				
Member Length (units)	4	5	6	7	8	4	5	6	7	8
Pattern	1313	1352	1425	1462	1537	1324	1352	1425	1462	1537
Moment Capacity (kip-ft)	658.8	678.3	854.1	879.4	1021.1	760.5	857.8	957.4	948.2	1085.0
$w_{D,truss}$ (plf)	328.55	328.55	328.55	328.55	328.55	328.55	328.55	328.55	328.55	328.55
$w_{D,deck}$/truss (plf)	264.45	264.45	264.45	264.45	264.45	264.45	264.45	264.45	264.45	264.45
$w_{D,roof}$/truss (plf)	140.04	140.04	140.04	140.04	140.04	140.04	140.04	140.04	140.04	140.04
Span (ft)	**72**	**73**	**82**	**84**	**90**	**78**	**83**	**87**	**87**	**93**
$M_{D,tot}$ (kip-ft)	475.02	488.3	616.13	646.55	742.21	557.48	631.25	693.56	693.56	792.52
$M_{max,LL}$ (kip-ft)	696.97	719.16	903.1	905.32	1068.4	786.25	882.09	1006.5	983.51	1127.5
H10-44: M_{LL} (kip-ft)	661	679.5	857.3	899.6	1032.8	775.7	878.4	965.1	965.1	1102.7

It is interesting to note that, in some cases, increasing member length has little or no effect on the maximum span of the bridge. This is due to the nature of the failure mode analysis for chord patterns. The 6-unit pattern and the 7-unit pattern with 4 pegs both have strength dominated by a combination mode with an identical count of members and connections. Because of this, increasing the member length to 7-units has no effect on increasing the strength of the bottommost chord, leading to no increase in moment capacity and no increase in maximum span. There is similarly very little difference between the 4-unit and 5-unit patterns with 3-peg connections.

Despite a lack of increase in strength and span, it is still advised to use the maximum length of member that is available for the construction of the Town lattice truss. While there is not always a strength increase when increasing the member length, there will always be a stiffness increase, which is desirable to reduce deflections in the span, while not increasing the material used in the bridge in any way.

Another point of note is that there is an obvious increase in capacity and span between trusses with 3-peg connections and trusses with 4-peg connections. Since connections strength seems to have such an impact on overall capacity, it is recommended that trusses be designed with 4-peg connections.

7.3 - Design methodology

The design procedure for the Town lattice truss will be based on all of the equations and relationships used to analyze a typical truss in Section 7.2. As opposed to defining the initial geometry of the truss, however, the required span and live load capacity will be defined in advance and the geometric properties will be varied to find a minimum solution.

Several properties must be decided before an iterative design procedure can begin. The member length shall be initially decided based on the available materials and fabrication tools. In addition, the number of pegs in the connections must be selected. Based on evidence from the example analysis, 4-peg connections are recommended as they can significantly increase the allowable span.

Finally, the wood to be used in the members and pegs must be known. If possible, material testing for strength and stiffness should be performed. If this is not possible, material properties such as specific gravity should be measured to allow for the estimation of properties.

The steps of the design process are:

Step 1: Define basic geometric properties
Step 2: Determine component properties
Step 3: Determine chord properties
Step 4: Determine live load capacity requirement
Step 5: Perform iterative design procedure to determine final truss geometry

Step 1: Define basic geometric properties

- joint spacing, s: 48” recommended
- member dimensions, b_c, d_c, b_w, d_w: 3” x 12” recommended for all members
- number of lines of chords members, N_c: 4 recommended
- number of pegs per chord connection, N_{peg}: 4 recommended
- peg diameter, D: 2” recommended

Step 2: Determine component properties

<u>Member properties</u>
Strength:

$$F_{m,\max} = F_t \cdot A_t = F_t \cdot b_c \cdot \left(d_c - 2 \cdot D\right)$$

Stiffness:

$$k_m = \frac{E \cdot A_c}{s} = \frac{E \cdot b_c \cdot d_c}{s}$$

<u>Connection properties</u>

Strength:
Single peg

Mode $\mathbf{I_d}$ $\quad ZI_d = 662.5 \cdot D \cdot t \cdot G_{dd}^{2.04}$ or $ZI_d = \dfrac{D \cdot t \cdot F_{ed}}{8}$

Mode $\mathbf{V_d}$ $\quad ZV_d = 538.8 \cdot D^2 \cdot G_{dd}^{0.84}$ or $ZV_d = 0.223 \cdot D^2 \cdot F_{ev}$

Connection strength

$$F_{m,\max} = N_{peg} \cdot \min(ZI_d, ZV_d)$$

Stiffness:

$$k_c = N_{peg} \cdot 0.2829 \cdot E^{0.1453} \cdot D^{1.436} \cdot k_b^{\ 0.8548}$$

E and k_b should be measured through testing of the peg and member materials, if possible. If not, a value for E can be estimated from tables or using the equation

$$E = 2.39 \cdot G_{12}^{\ 0.7} \cdot 10^6 \text{ psi}$$

and an average value for k_b of 20000 lb/in can be used.

Step 3: Determine chord properties

Strength:
The failure modes for the appropriate pattern length, given in Table 7.8, should be used to determine failure load for the pattern options. The load for each failure mode is calculated by multiplying the number of member failures and the number of connection failures of a specific mode by the member and connection strengths, respectively, and summing the result. Thus, for pattern length 4, the chord failure load for each pattern would be determined as:

$$F_{ch,\max,4-1313} = \min(0F_{m,\max} + 6F_{c,\max}, 1F_{m,\max} + 2F_{c,\max}, 2F_{m,\max} + 0F_{c,\max})$$

$$F_{ch,\max,4-1324} = \min(0F_{m,\max} + 5F_{c,\max}, 1F_{m,\max} + 2F_{c,\max}, 2F_{m,\max} + 1F_{c,\max}, 3F_{m,\max} + 0F_{c,\max})$$

The pattern with the higher failure load should be selected for use in the truss. Thus, for pattern length 4:

$$F_{ch,\max,4} = \max(F_{ch,\max,4-1313}, F_{ch,\max,4-1324})$$

Table 7.8 - Highest strength patterns with lengths from 4 to 8 units

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c
4	1 3 1 3	0 6	1 2	2 0	
4	1 3 2 4	0 5	1 3	2 1	3 0
5	1 3 5 2	0 6	1 3	2 1	3 0
6	1 4 1 4	0 9	1 3	2 0	
6	1 4 2 5	0 8	1 4	2 1	3 0
7	1 4 7 3	0 9	1 4	2 1	3 0
7	1 4 6 2	0 8	1 5	2 1	3 0
8	1 5 1 5	0 12	1 4	2 0	
8	1 5 2 6	0 11	1 5	2 1	3 0
8	1 5 3 7	0 10	1 6	2 2	3 0

Stiffness:
The stiffness factor for the selected pattern should be calculated based on component stiffnesses using the equation:

$$\alpha = C_1 \cdot k_m^{C_2} \cdot k_c^{C_3}$$

and the coefficients from Table 7.9.

Table 7.9 – Stiffness factor coefficients for highest strength patterns with lengths from 4 to 8 units

N	Pattern	C_1	C_2	C_3
4	1 3 1 3	0.8052	-0.7610	0.7818
4	1 3 2 4	0.8175	-0.7716	0.7917
5	1 3 5 2	0.8992	-0.6893	0.7131
6	1 4 1 4	0.9180	-0.6028	0.6288
6	1 4 2 5	0.9342	-0.6094	0.6352
7	1 4 7 3	0.9388	-0.5387	0.5647
7	1 4 6 2	0.9545	-0.5476	0.5737
8	1 5 1 5	0.9228	-0.4712	0.4967
8	1 5 2 6	0.9359	-0.4754	0.5007
8	1 5 3 7	0.9510	-0.4845	0.5100

Step 4: Determine live load capacity requirement

The goal of the design is to determine a truss cross-section that can support load over a given span. When the desired span is known, it is possible to immediately determine the required live load moment capacity of the truss based on known applied loading. As was developed in Chapter 3, the recommended live loading is a combination of an AASHTO H10-44 truck loading and general pedestrian loading. This produced a live load design table as shown in Table 7.10.

Table 7.10 - Design moment and end shear for single-lane simply-supported beam bridge based on AASHTO H10-44 loading. Letters indicate dominant load type: (a) Pedestrian loading; (b) Lane load; (c) Design truck

Span (ft)	Moment (kip-ft)	End Shear (kip)	Span (ft)	Moment (kip-ft)	End Shear (kip)	Span (ft)	Moment (kip-ft)	End Shear (kip)
1	4.0 (c)	16.0 (c)	34	147.4 (a)	18.4 (b)	67	572.4 (a)	34.2 (a)
2	8.0 (c)	16.0 (c)	35	156.2 (a)	18.6 (b)	68	589.6 (a)	34.7 (a)
3	12.0 (c)	16.0 (c)	36	165.2 (a)	18.8 (b)	69	607.0 (a)	35.2 (a)
4	16.0 (c)	16.0 (c)	37	174.6 (a)	18.9 (b)	70	624.8 (a)	35.7 (a)
5	20.0 (c)	16.0 (c)	38	184.1 (a)	19.4 (a)	71	642.7 (a)	36.2 (a)
6	24.0 (c)	16.0 (c)	39	193.9 (a)	19.9 (a)	72	661.0 (a)	36.7 (a)
7	28.0 (c)	16.0 (c)	40	204.0 (a)	20.4 (a)	73	679.5 (a)	37.2 (a)
8	32.0 (c)	16.0 (c)	41	214.3 (a)	20.9 (a)	74	698.2 (a)	37.7 (a)
9	36.0 (c)	16.0 (c)	42	224.9 (a)	21.4 (a)	75	717.2 (a)	38.3 (a)
10	40.0 (c)	16.0 (c)	43	235.8 (a)	21.9 (a)	76	736.4 (a)	38.8 (a)
11	44.0 (c)	16.0 (c)	44	246.8 (a)	22.4 (a)	77	756.0 (a)	39.3 (a)
12	48.0 (c)	16.0 (c)	45	258.2 (a)	23.0 (a)	78	775.7 (a)	39.8 (a)
13	52.0 (c)	16.0 (c)	46	269.8 (a)	23.5 (a)	79	795.7 (a)	40.3 (a)
14	56.0 (c)	16.0 (c)	47	281.7 (a)	24.0 (a)	80	816.0 (a)	40.8 (a)
15	60.0 (c)	16.3 (c)	48	293.8 (a)	24.5 (a)	81	836.5 (a)	41.3 (a)
16	64.0 (c)	16.5 (c)	49	306.1 (a)	25.0 (a)	82	857.3 (a)	41.8 (a)
17	68.0 (c)	16.7 (c)	50	318.8 (a)	25.5 (a)	83	878.4 (a)	42.3 (a)
18	72.0 (c)	16.9 (c)	51	331.6 (a)	26.0 (a)	84	899.6 (a)	42.8 (a)
19	76.0 (c)	17.1 (c)	52	344.8 (a)	26.5 (a)	85	921.2 (a)	43.4 (a)
20	80.0 (c)	17.2 (c)	53	358.2 (a)	27.0 (a)	86	943.0 (a)	43.9 (a)
21	84.0 (c)	17.3 (c)	54	371.8 (a)	27.5 (a)	87	965.1 (a)	44.4 (a)
22	88.0 (c)	17.5 (c)	55	385.7 (a)	28.1 (a)	88	987.4 (a)	44.9 (a)
23	92.0 (c)	17.6 (c)	56	399.8 (a)	28.6 (a)	89	1009.9 (a)	45.4 (a)
24	96.0 (c)	17.7 (c)	57	414.3 (a)	29.1 (a)	90	1032.8 (a)	45.9 (a)
25	100.0 (c)	17.8 (c)	58	428.9 (a)	29.6 (a)	91	1055.8 (a)	46.4 (a)
26	104.0 (c)	17.8 (c)	59	443.8 (a)	30.1 (a)	92	1079.2 (a)	46.9 (a)
27	108.5 (c)	17.9 (c)	60	459.0 (a)	30.6 (a)	93	1102.7 (a)	47.4 (a)
28	113.4 (c)	18.0 (c)	61	474.4 (a)	31.1 (a)	94	1126.6 (a)	47.9 (a)
29	118.4 (c)	18.1 (c)	62	490.1 (a)	31.6 (a)	95	1150.7 (a)	48.5 (a)
30	123.3 (c)	18.1 (c)	63	506.1 (a)	32.1 (a)	96	1175.0 (a)	49.0 (a)
31	128.3 (c)	18.2 (c)	64	522.2 (a)	32.6 (a)	97	1199.6 (a)	49.5 (a)
32	133.2 (c)	18.3 (c)	65	538.7 (a)	33.2 (a)	98	1224.5 (a)	50.0 (a)
33	138.9 (a)	18.3 (c)	66	555.4 (a)	33.7 (a)	99	1249.6 (a)	50.5 (a)
						100	1275.0 (a)	51.0 (a)

At the upper end of the range of spans shown, the live loading is dominated by the pedestrian loading, which is based on the AASHTO recommended uniformly distributed pressure of 0.085 kip/ft^2 exerted over an assumed 12 ft width. This yields a design live load moment of:

$$M_{LL} = \frac{0.085 \cdot 12 \cdot L^2}{8} = 0.1275 \cdot L^2$$

where L is the span of the bridge.

This pedestrian loading is high and is based on a bridge fully loaded with pedestrians. For a remote bridge where it can be guaranteed that this will not happen, it might be possible to reduce this requirement, but for any bridge that is near a community, it is possible that this loading may occur at some point, most likely when the bridge is first opened.

Step 5: Perform iterative design procedure to determine final truss geometry

The main parameter that can be adjusted to affect the moment capacity of the truss is the overall truss height. This height will be controlled by two other parameters: the number of rows of joints, N, and the web angle, μ. To allow for the use as a covered bridge, the truss must be high enough to allow for truck clearance, but not so high that the truss loses lateral rigidity. Based on existing bridges, a range of truss heights from 12.5' to 16.5' is recommended.

For every iteration in truss height, the moment capacity and the moment from dead load must both be recalculated. The difference between the two will be the live load capacity of the bridge, which can be compared with the required love load capacity. The objective is to find a design that just exceeds the requirement.

Iteration Step 1

Select values for number of lines of joints, N, and web angle, μ.

Iteration Step 2

Calculate truss height, H

$$H = \frac{s}{2} \cdot (N-1) \cdot \tan\mu$$

and height between rows of joints, h

$$h = \frac{s}{2} \cdot \tan\mu$$

Iteration Step 3

Determine moment capacity of truss.

$$M_{\max} = F_{LB} \cdot (H-c) + F_{UB} \cdot (H-h-c) + F_{LT} \cdot (c-h) + F_{UT} \cdot (c)$$

where

$$F_{UB} = \left(\frac{H-c-h}{H-c}\right) \cdot F_{CH,\max}$$

$$F_{LT} = \frac{1}{\alpha}\left(\frac{c-h}{H-c}\right) \cdot F_{CH,\max}$$

$$F_{UT} = \frac{1}{\alpha}\left(\frac{c}{H-c}\right) \cdot F_{CH,\max}$$

and $$c = \frac{h - \alpha h + 2\alpha H}{2+2\alpha}$$

Iteration Step 4

Determine design moment from dead load

$$M_{\max,DL} = \frac{w_{D,tot} \cdot L^2}{8}$$

where

$$w_{D,tot} = 2 \cdot w_{D,truss} + w_{D,deck} + w_{D,roof}$$

$$w_{D,truss} = \frac{\gamma}{s} \cdot \left(4 \cdot b_c \cdot d_c \cdot s \cdot N_c + 2 \cdot b_w \cdot d_w \cdot \left(\frac{H}{\sin \mu} + 2 \cdot L_e \right) \right)$$

$$w_{D,deck} = \left(\frac{1}{k_{dcb} \cdot s} \cdot b_{dcb} \cdot d_{dcb} \cdot L_{dcb} + d_d \cdot w_d \right) \cdot \gamma$$ with a typical value of

$w_{D,deck} = 528.9$ lb/ft

$$w_{D,roof} = \left(\frac{1}{k_{rcb} \cdot s} \cdot b_{rcb} \cdot d_{rcb} \cdot L_{rcb} + t_r \cdot \frac{L_{rcb}}{\cos \theta} \right) \cdot \gamma$$ with a typical value of

$w_{D,roof} = 281.1$ lb/ft

Iteration Step 5

Determine live load capacity of bridge and compare with required live load capacity

$$M_{\max,LL} = 2 \cdot C_d \cdot M_{\max,truss} - M_{\max,DL}$$

Iteration Step 6

If $M_{\max,LL} >> M_{LL}$, decrease N or μ and return to Step 2

If $M_{\max,LL} < M_{LL}$, increase N or μ and return to Step 2

If Mmax,LL is only slightly greater than the MLL, the iterative procedure may be complete. Values should be adjusted slightly to confirm that no better solution exists. If none are found then the iterative procedure is finished and the current values for N and μ should be selected as final design parameters.

7.4 - Example designs with comparison and assessment

Sets of designs were developed for different member lengths using the same base values used in the example analysis in Section 7.2. 4-peg connections are used throughout. Results are presented in Table 7.11 through Table 7.15.

Table 7.11 - Design results for 4-unit member length with 4-peg connections

Member Length (units)	4	4	4	4
Pattern	1324	1324	1324	1324
Chord Design Strength (lbs)	31688	31688	31688	31688
Chord Stiffness Factor	0.159	0.159	0.159	0.159
Span, L (ft)	70	75	80	85
Web angle, μ (deg)	51	49	53	52
# lines of joints, N	6	7	7	8
Height (in)	148.2	165.7	191.1	215.0
Height (ft)	12.35	13.80	15.92	17.92
h (in)	29.6	27.6	31.8	30.7
Neutral Axis Depth	31.11	32.77	37.81	40.69
Fut (lbs)	52855	49061	49061	46421
Flt (lbs)	2499	7730	7730	11372
Fub (lbs)	23666	25104	25104	26105
Moment Capacity M_{max} (kip-ft)	618.9	708.4	817.2	939.7
$w_{D,t}$ (lb/ft)	308.7	323.7	334.0	351.5
$w_{D,deck}$/truss (lb/ft)	264.5	264.5	264.5	264.5
$w_{D,roof}$/truss (lb/ft)	140.0	140.0	140.0	140.0
$M_{max,d}$/truss (kip-ft)	436.8	512.0	590.8	682.8
Live Load Capacity, $M_{LL,max}$ (kip-ft)	673.7	747.1	861.5	983.6
H10-44 Live Loading, M_{LL} (kip-ft)	624.8	717.2	816	921.2

Table 7.12 - Design results for 5-unit member length with 4-peg connections

Member Length (units)	5	5	5	5
Pattern	1352	1352	1352	1352
Chord Design Strength (lbs)	36533	36533	36533	36533
Chord Stiffness Factor	0.223	0.223	0.223	0.223
Span, L (ft)	75	80	85	90
Web angle, μ (deg)	52	49	53	51
# lines of joints, N	6	7	7	8
Height (in)	153.6	165.7	191.1	207.5
Height (ft)	12.80	13.80	15.92	17.29
h (in)	30.7	27.6	31.8	29.6
Neutral Axis Depth	37.78	38.99	44.98	47.27
Fut (lbs)	53394	50389	50389	48293
Flt (lbs)	9982	14713	14713	18013
Fub (lbs)	26842	28569	28569	29774
Moment Capacity M_{max} (kip-ft)	716.9	799.1	921.8	1028.3
$w_{D,t}$ (lb/ft)	310.9	323.7	334.0	348.4
$w_{D,deck}$/truss (lb/ft)	264.5	264.5	264.5	264.5
$w_{D,roof}$/truss (lb/ft)	140.0	140.0	140.0	140.0
$M_{max,d}$/truss (kip-ft)	503.0	582.6	667.0	762.3
Live Load Capacity, $M_{LL,max}$ (kip-ft)	786.2	832.7	970.7	1046.1
H10-44 Live Loading, M_{LL} (kip-ft)	717.2	816	921.2	1032.8

Table 7.13 - Design results for 6-unit member length with 4-peg connections

Member Length (units)	6	6	6	6
Pattern	1425	1425	1425	1425
Chord Design Strength (lbs)	41378	41378	41378	41378
Chord Stiffness Factor	0.288	0.288	0.288	0.288
Span, L (ft)	80	85	90	95
Web angle, μ (deg)	52	50	53	51
# lines of joints, N	6	7	7	8
Height (in)	153.6	171.6	191.1	207.5
Height (ft)	12.80	14.30	15.92	17.29
h (in)	30.7	28.6	31.8	29.6
Neutral Axis Depth	42.86	46.30	51.56	54.61
Fut (lbs)	55546	53036	53036	51281
Flt (lbs)	15731	20275	20275	23451
Fub (lbs)	29899	31933	31933	33355
Moment Capacity M_{max} (kip-ft)	795.5	924.0	1028.9	1151.7
$w_{D,t}$ (lb/ft)	310.9	326.1	334.0	348.4
$w_{D,deck}$/truss (lb/ft)	264.5	264.5	264.5	264.5
$w_{D,roof}$/truss (lb/ft)	140.0	140.0	140.0	140.0
$M_{max,d}$/truss (kip-ft)	572.3	659.8	747.7	849.4
Live Load Capacity, $M_{LL,max}$ (kip-ft)	844.1	990.4	1076.7	1180.5
H10-44 Live Loading, M_{LL} (kip-ft)	816.0	921.2	1032.8	1150.7

Table 7.14 - Design results for 7-unit member length with 4-peg connections

Member Length (units)	7	7	7	7
Pattern	1473	1473	1473	1473
Chord Design Strength (lbs)	41378	41378	41378	41378
Chord Stiffness Factor	0.345	0.345	0.345	0.345
Span, L (ft)	80	85	90	95
Web angle, μ (deg)	52	50	53	51
# lines of joints, N	6	7	7	8
Height (in)	153.6	171.6	191.1	207.5
Height (ft)	12.80	14.30	15.92	17.29
h (in)	30.7	28.6	31.8	29.6
Neutral Axis Depth	46.86	50.97	56.75	60.41
Fut (lbs)	52693	50698	50698	49301
Flt (lbs)	18153	22247	22247	25114
Fub (lbs)	29468	31568	31568	33038
Moment Capacity M_{max} (kip-ft)	784.9	914.9	1018.8	1142.9
$w_{D,t}$ (lb/ft)	310.9	326.1	334.0	348.4
$w_{D,deck}$/truss (lb/ft)	264.5	264.5	264.5	264.5
$w_{D,roof}$/truss (lb/ft)	140.0	140.0	140.0	140.0
$M_{max,d}$/truss (kip-ft)	572.3	659.8	747.7	849.4
Live Load Capacity, $M_{LL,max}$ (kip-ft)	817.6	967.8	1051.5	1158.5
H10-44 Live Loading, M_{LL} (kip-ft)	816.0	921.2	1032.8	1150.7

Table 7.15 - Design results for 8-unit member length with 4-peg connections

Member Length (units)	8	8	8	8
Pattern	1537	1537	1537	1537
Chord Design Strength (lbs)	47715	47715	47715	47715
Chord Stiffness Factor	0.396	0.396	0.396	0.396
Span, *L* (ft)	90	95	100	105
Web angle, μ (deg)	49	52	50	53
# lines of joints, *N*	7	7	8	8
Height (in)	165.7	184.3	200.2	222.9
Height (ft)	13.80	15.36	16.68	18.58
h (in)	27.6	30.7	28.6	31.8
Neutral Axis Depth	52.96	58.92	62.98	70.13
Fut (lbs)	56632	56632	55301	55301
Flt (lbs)	27108	27108	30185	30185
Fub (lbs)	36025	36025	37771	37771
Moment Capacity M_{max} (kip-ft)	1010.7	1124.6	1264.3	1407.8
$w_{D,t}$ (lb/ft)	323.7	331.2	345.5	354.8
$w_{D,deck}$/truss (lb/ft)	264.5	264.5	264.5	264.5
$w_{D,roof}$/truss (lb/ft)	140.0	140.0	140.0	140.0
$M_{max,d}$/truss (kip-ft)	737.3	830.0	937.5	1046.4
Live Load Capacity, $M_{LL,max}$ (kip-ft)	1052.3	1151.5	1285.8	1426.9
H10-44 Live Loading, M_{LL} (kip-ft)	1032.8	1150.7	1275.0	1405.7

The design results were specifically selected such that all heights are close to, or within, the range of recommended heights for the Town lattice truss to be used as part of a covered bridge, specifically 12.5’ to 16.5’. The efficiency of material use is evaluated by plotting span as a function of volume of material used. These results are shown in Figure 7.6 along with similar curves from other timber bridge systems, which were described and analyzed in Chapter 3.

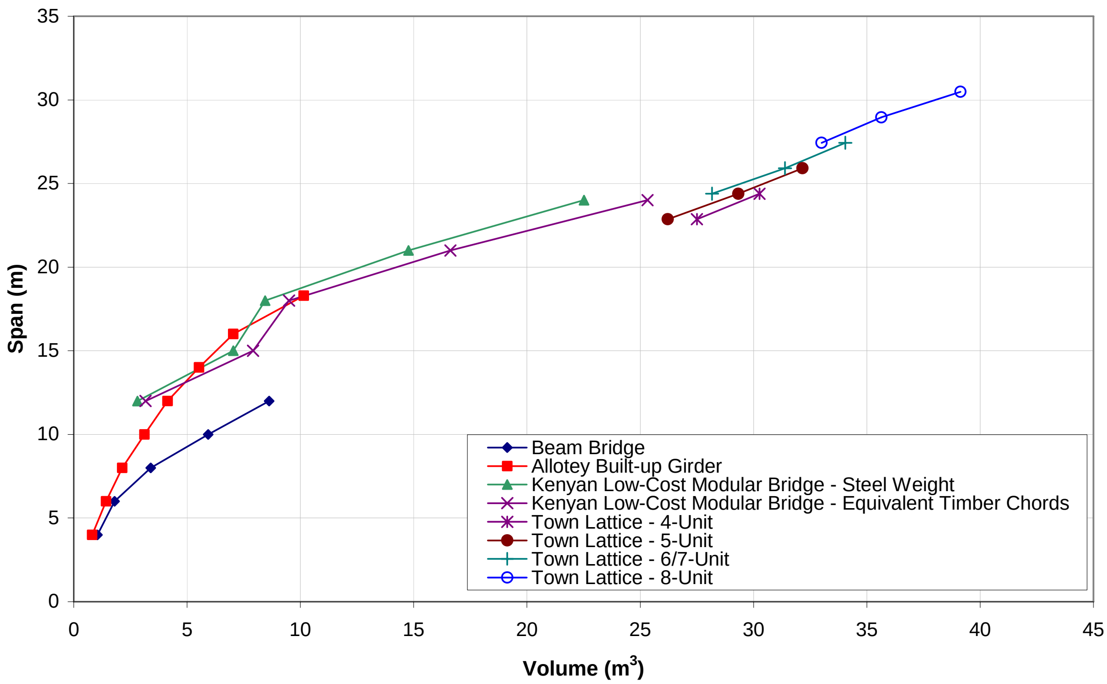


Figure 7.6 - Comparison of span as a function of volume of material for timber bridge types

The results plotted in Figure 7.6 show that the Town lattice truss system follows a similar efficiency curve to the Allotey Built-up Girder and the Kenyan Low-Cost Modular Bridge, while offering greater spans than either of the other two systems. This validates the Town lattice truss as a potential appropriate bridge technology by demonstrating competitive functionally with other timber bridge systems that have been proposed for use in developing countries.

One point of note is that the maximum spans found for the example designs are significantly less than many of the spans seen in existing Town lattice truss bridges. This difference can be primarily explained by a difference in assumed loading. The bridges designed herein assume a potential full pedestrian loading, which is highly conservative for many bridges where such a loading is unlikely to occur. Most bridges in the United States do not need to be designed to support a full pedestrian loading unless they are specifically intended for such a use. Timber covered bridges, typically used in more remote locations, are unlikely to ever see significant pedestrian loading, and are thus assessed for supporting vehicle loading only, often at a reduced tonnage. In the developing world, however, foot traffic is much more prevalent and a full pedestrian loading is more likely to occur. If such a loading can be reasonably considered to be unlikely, it will be possible to reduce the required live load capacity of the bridges, potentially increasing their maximum allowable span.

To illustrate this difference, a second set of design results were created for 8-unit patterns with H10-44 loading and no pedestrian loading requirement. Results are given in Table 7.16.

Table 7.16 - Design results for 8-unit member length with 4-peg connections supporting H10-44 loading only

Member Length (units)	8	8	8	8
Pattern	1537	1537	1537	1537
Chord Design Strength (lbs)	47715	47715	47715	47715
Chord Stiffness Factor	0.396	0.396	0.396	0.396
Span, L (ft)	100	105	110	115
Web angle, μ (deg)	49	51	54	51
# lines of joints, N	7	7	7	8
Height (in)	165.7	177.8	198.2	207.5
Height (ft)	13.80	14.82	16.52	17.29
h (in)	27.6	29.6	33.0	29.6
Neutral Axis Depth	52.96	56.85	63.36	65.26
Fut (lbs)	56632	56632	56632	55301
Flt (lbs)	27108	27108	27108	30185
Fub (lbs)	36025	36025	36025	37771
Moment Capacity M_{max} (kip-ft)	1010.7	1085.0	1209.3	1310.1
$w_{D,t}$ (lb/ft)	323.7	328.6	337.0	348.4
$w_{D,deck}$/truss (lb/ft)	264.5	264.5	264.5	264.5
$w_{D,roof}$/truss (lb/ft)	140.0	140.0	140.0	140.0
$M_{max,d}$/truss (kip-ft)	910.2	1010.2	1121.5	1244.7
Live Load Capacity, $M_{LL,max}$ (kip-ft)	706.4	692.0	780.3	785.9
H10-44 Live Loading, M_{LL} (kip-ft)	625.0	677.3	731.5	787.8

This second set of design results show an increase in allowable span of 10' when loading is restricted to vehicular loading only. This final span is still less than many of the spans seen in existing lattice truss bridges, however the longest of these bridges have posted weight limits of as low as 3 tons. Posting weight limits is not considered an adequate precaution for bridges in developing countries, and no vehicle loading less than H10-44 is recommended.

7.5 - Summary

The moment capacity of a Town lattice truss can now be easily determined using relationships developed in this work. Component properties can be determined using rules developed in Chapter 5. Component properties are combined together to yield chord properties, as described in Chapter 6. These chord properties are then considered within the Town lattice truss cross-section and used to determine the moment capacity of the truss. This is the first work to address all of these components of the system and represents a clear advancement on the understanding of Town lattice truss bridges.

While this work represents a significant contribution, there is a need for further validation and refinement of the details. Since very little has ever been done to better understand these bridges, there is a lack of experimental data on the strength and stiffness of the components of the bridge and the failure behaviour of the chords and overall truss. Further material testing is recommended, particularly for timber species that might be used in developing countries where this bridge offers an appropriate solution. Overall testing of Town lattice truss chords and trusses is also recommended to assess the validity of the assumptions made in the development of procedures for determining moment capacity.

All of the procedures developed herein have been consolidated into a simple design methodology for the determination of the appropriate Town lattice truss cross-section for a given span. This methodology is simple enough that it could be used by an engineer in the design of rural road bridges for developing countries. The designs that are generated include significant factors of safety and assume conservative loading for the bridges, and are therefore considered likely to be safe for use on rural roads while not being over-designed to an inappropriate level. A comparison of designs generated from the methodology with existing Town lattice truss bridges also suggests that these designs are reasonable.

Chapter 8 – Summary and Future Work

8.1 - Summary

The original objective of this work was to determine an appropriate bridge technology for use in the Tshumbe Diocese. The Town lattice truss is proposed and developed as an appropriate technology, although there are a number of aspects that must be addressed to make the system ready for implementation in the field.

The work was not approached as a simple design task, but rather as an opportunity to explore the realm of appropriate bridge technology and to contribute to the understanding of the structural behaviour of a historic timber bridge system. In these realms, a number of conclusions and results are proposed.

In identifying an appropriate bridge technology, it was first important to understand what makes a bridge technology appropriate. It was concluded that, first, an appropriate bridge technology must have the required characteristics that make any technology appropriate. Based on the framework established in Chapter 2 of this work, there must be a need and desire for the technology, the technology must be designed or selected to have functional adequacy, economic feasibility, and sustainability, and the technology should cause no serious adverse environmental effects. These requirements are similar to requirements that should exist for all technologies, even those used in the developed world, although there are some specific nuances that apply to work in the developing world that must be considered.

An appropriate bridge technology must also be developed within, and contribute to, an overarching plan for general rural transport development that is focused on the poor. An appropriate bridge must fit within the infrastructure network for which it is designed and satisfy the needs of those who will use it most, while not drawing away significant resources from other aspects of transport development.

Finally, the nature of bridges makes it such that, because of their scale, they can have significant indirect economic benefits if they incorporate consideration of the desirable characteristics of appropriate technology. In particular, the use of local materials, labour-based methods with local labour, and training and empowering the local people can all contribute to broader economic effects, and their simultaneous application will compound these benefits.

With these characteristics in mind, the Town lattice truss was identified as a potential appropriate technology, and a number of its characteristics were highlighted to illustrate their contribution to its appropriateness. Two of the unique characteristics of the Town lattice truss that are crucial to its appropriateness are the wooden pegged connections and the chord structure. Neither of these elements has ever before been the subject of significant research and improving the understanding of their nature and behaviour is one of the major contributions of this work.

Wooden pegged connections offer a significant advantage to the Town lattice truss in terms of appropriateness in timber-rich rural areas. The use of fully wooden connections eliminates the dependence on metal, which must be sourced non-locally, while increasing the use of local materials. The method of assembly for these connections is simple and labour-based, fostering employment of local unskilled workers while providing experience and knowledge in simple woodworking.

The strength of wooden pegged connections has been previously studied to a limited extent, primarily in relation to their use in mortise and tenon connections. These connections differ from those in the Town lattice truss in terms of member thickness, peg diameter, and grain orientation. Results from other research were compiled and used to extrapolate design strength equations for wooden pegged connections as used in the Town lattice truss. Connection strength was found to be dominated by failure of the wooden peg, either in bearing or shear. More data is needed for these particular modes to improve confidence levels in predicting strength.

The stiffness of wooden pegged connections was also addressed. Equations were developed that allow for the estimation of connection stiffness based on peg diameter, member thickness, and peg and material stiffness properties. The procedure is limited to an estimation of stiffness due to a combination of highly variable material properties and a lack of experimental data.

The evaluation of the chord structure, the second unique appropriate characteristic of the Town lattice truss, is the most significant contribution of this work. A comprehensive study of this particular aspect of the Town lattice structure has never been conducted. When new Town lattice truss bridges are erected, typically as replacements for bridges that have been lost due to flood or fire, there has been no clear guidance on the selection of chord termination pattern. Patterns are selected based on those used in the bridge that is being replaced or intuition about how the pattern is expected to behave. A better understanding of the behaviour of chord termination patterns is important to eliminate arbitrary decisions that can have a significant impact on the overall performance of the Town lattice truss.

Chord termination patterns were catalogued, both as a set of patterns seen in use in Town lattice truss bridges and as a comprehensive set of patterns that could theoretically be used. The set of possible patterns was systematically assessed for strength based on a failure mode analysis. The resulting set of modes allowed for the identification of the best patterns for use in the design and construction of new Town lattice truss structures. Patterns were also analyzed to determine their effective axial stiffness, a property that is needed in the determination of moment capacity and could also be used in predicting deflection in Town lattice truss bridges under service loading.

The final significant contribution of the work is a design methodology for Town lattice trusses based on the component properties studied herein. This simple methodology allows for the determination of truss geometry for a desired span with a given design loading. The methodology is appropriate for the design of bridges both by engineers working on rural transport in the developing world and by transportation agencies in North America that have authority over timber covered bridges.

Selected results from the design procedure have been simplified and compiled into a draft design guide, which is included as Appendix F. This design guide presents an overview of a number of aspects of truss layout, fabrication and erection, and is intended to introduce and facilitate the adoption of the technology in developing countries.

8.2 - Future work

While this work offers significant contributions to the understanding of the behaviour of Town lattice truss bridges, more work is needed. In particular, there is a significant need for experimental data regarding the behaviour of the trusses and their components. Experimental testing in a variety of areas would help to calibrate analytical models and verify assumptions.

Component strength and stiffness equations developed and presented in this work are largely based on research focused on mortise and tenon connections. Only one known experimental study, of limited scope, has specifically focused on Town lattice truss connections. In particular, there is a need for data regarding the behaviour of wooden pegs in bearing and shear, as these are the modes that have been found to dominate in the failure of wooden pegged connections. Such tests should incorporate a variety of wood species, including tropical woods. In particular, for the results to be most applicable in developing countries, local wood species from those countries should be tested.

Chord structures and full trusses should also be tested to failure to verify assumption about failure modes and the distribution of forces. The effect of chord termination pattern, material relative stiffness and strength, and minor fabrication errors should all be assessed. One of the major challenges in such testing will be the application of loading that simulates realistic conditions at failure.

Finally, while the design results presented in this work only considered Town lattice trusses as they could be used in covered bridges, there is the potential for the truss to be competitive with other timber bridge systems in shorter spans if used in a pony-truss style, with trusses extending above the sides of the bridge but not connected by an overhead structure. The moment capacity design will follow the same methodology presented in this work, although a number of additional design details, such as a method of lateral bracing, would need to be developed to allow for implementation.

Appendix A - Structural Testing of Wood Samples from Tshumbe Diocese of the Democratic Republic of Congo

Five specimens of typical wood were provided by Bishop N. Djomo of the Tshumbe Diocese of the Democratic Republic of Congo. The five types of wood were identified as: Dihake, Oleko, Okolongo, Olondo, and Olongo. In addition, a sample of Douglas Fir was used to check the test results with published values.

Specimens

Provided wood samples were of varying sizes. Samples were cut into test specimens such that at least six specimens would be created for each type of wood. The resulting geometry for the specimens was a square cross-section with a depth and width of 17/32 in and a length of 12 in.

Testing Apparatus

An Instron testing machine (model 1331) was used for the testing. An Instron three-point bending apparatus, specifically designed for the machine, was used. A photograph of the test setup is shown in Figure A.1.

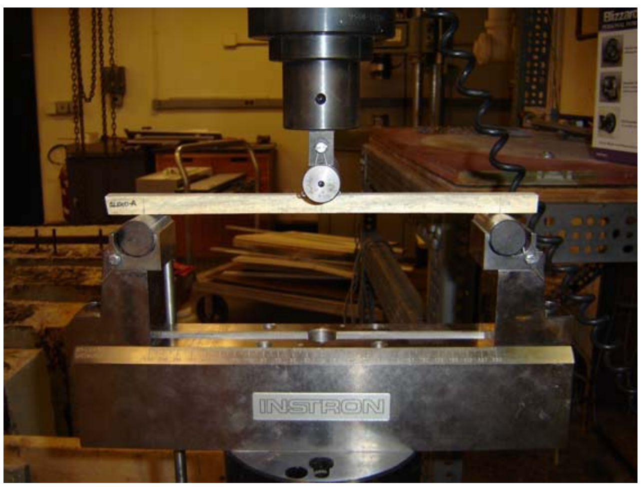


Figure A.1 - Three-point bending test setup

A 50kN load cell was used for two tests (Sample A and Sample B) and a 10kN load cell was used for all other tests. This was done because the 10 kN load cell offers a better resolution and was found to have sufficient capacity.

Testing Procedure

Specimens were set in the apparatus to have a clear span between supports of 9.75 in. The testing machine was adjusted to have contact but no load. Load was then applied by applying a constant rate of displacement of the center loading point of 0.05 in/min. This was considered to be slow enough to be modeled as a quasi-static load.

Load and position were outputted to a data acquisition system at a rate of 2 readings per second.

Converting to Mechanical Properties

The major two mechanical properties of interest are the modulus of elasticity (E) and the failure stress of modulus of rupture (σ_f). The modulus of elasticity relates to the linear stiffness of the wood and is obtained by relating displacement to load. The failure stress represents the strength of the wood and is calculated based on the failure load and the geometry of the cross-section.

Modulus of Elasticity
The relationship between load and displacement for a simply-supported beam with a point load at mid-span is known to be

$$F = \frac{48 \cdot E \cdot I}{L^3} \cdot u$$

giving $k = \frac{48 \cdot E \cdot I}{L^3}$

where k is the slope of the linear portion of the load-deflection curve, I is the moment of inertia of the cross-section, and L is the clear span of the structure. This can be rearranged to solve for the modulus of elasticity of the material as:

$$E = \frac{k \cdot L^3}{48 \cdot I}$$

The specific geometry used in the testing has:

$$I = \frac{1}{12}(17/32)^4 = 6.638 \cdot 10^{-3} \text{ in}^4$$

and $L = 9.75$ in

giving $E = 2909.1 \cdot k$

Modulus of Rupture
A concentrated point load at the center of a span creates a maximum internal moment at mid-span with an amplitude of:

$$M = \frac{F \cdot L}{4}$$

The maximum stress in a rectangular cross-section can be related to the moment as:

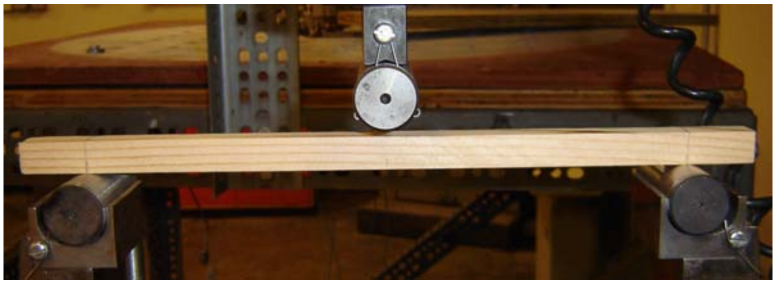
$$\sigma = \frac{M \cdot d}{2 \cdot I}$$

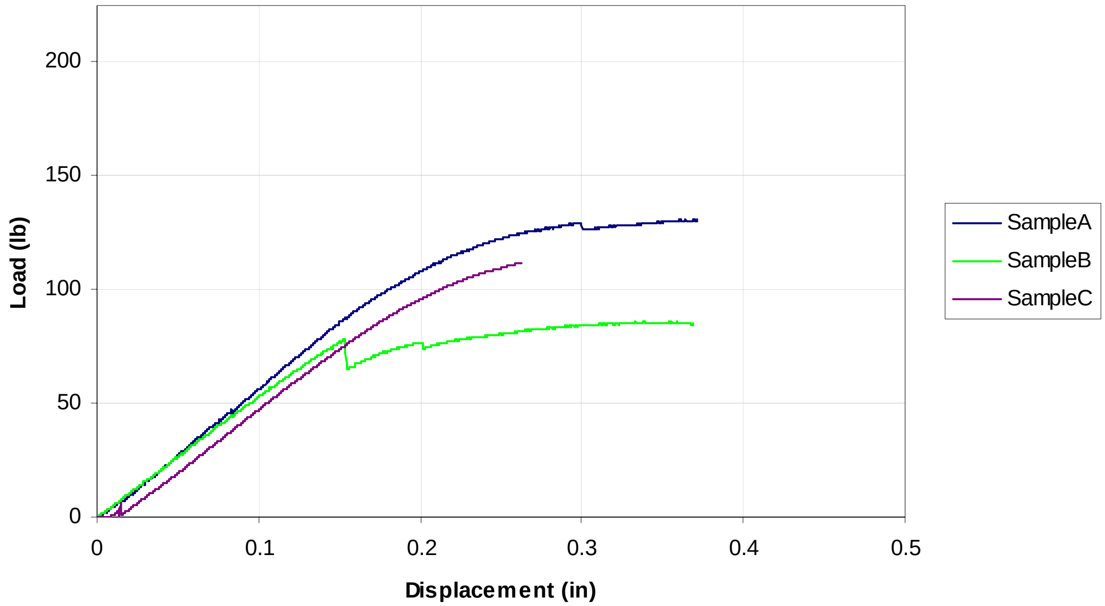
where d is the depth of the cross-section.

The specific geometry used in the testing yields:

$$\sigma = 48.77 \cdot F$$

Sample Wood – Douglas Fir





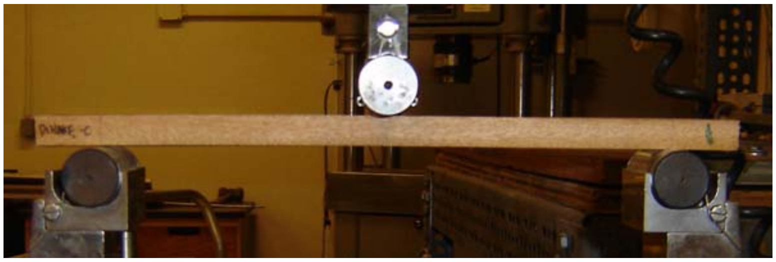
	k (lb/in)	E (10^3 psi)	F_{max} (lb)	σ_{max} (psi)
SampleA-Douglas Fir	586.83	1707.2	130.56	6367
SampleB-Douglas Fir	515.31	1499.1	85.12	4152
SampleC-Douglas Fir	555.88	1617.1	105.28	5135
Average		1607.8		5218

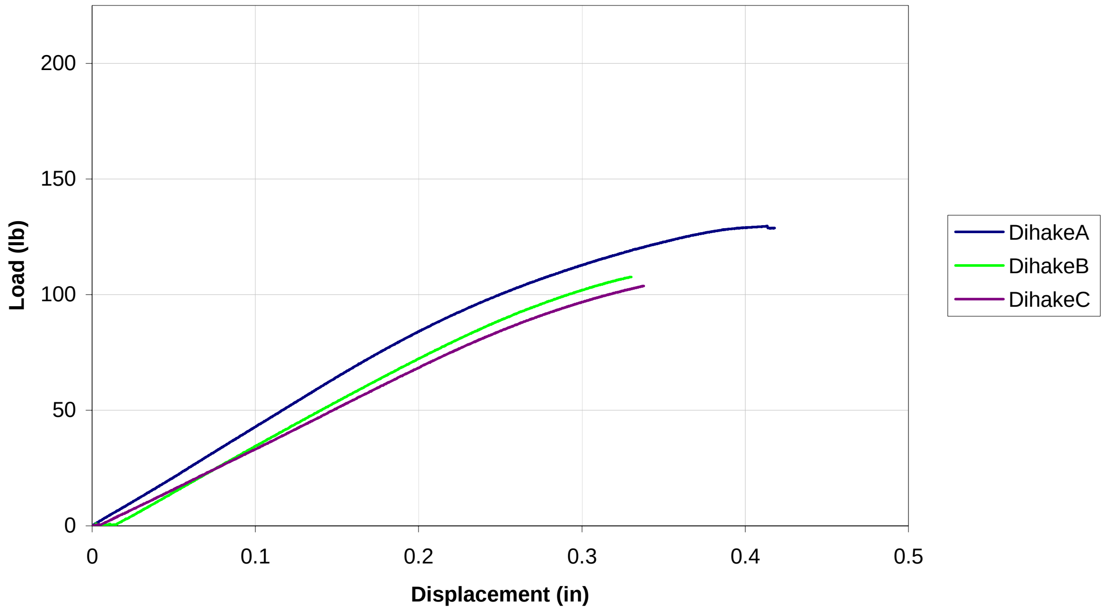
Published Values
Values taken from Wood Handbook (Forest Products Laboratory 1999) for species Douglas Fir - Coast

E = 1560 $\cdot 10^3$ psi (green) to 1950 $\cdot 10^3$ psi (12% MC)

Modulus of rupture = 7700 psi (green) to 12400 psi (12% MC)

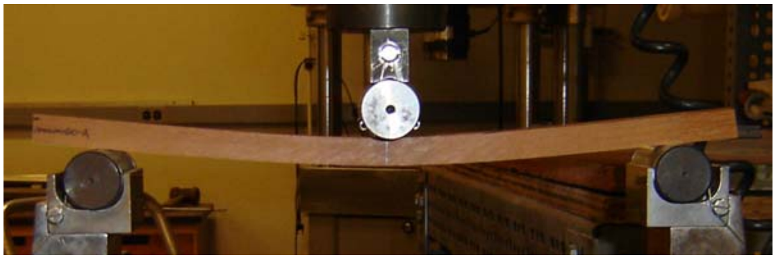
Dihake

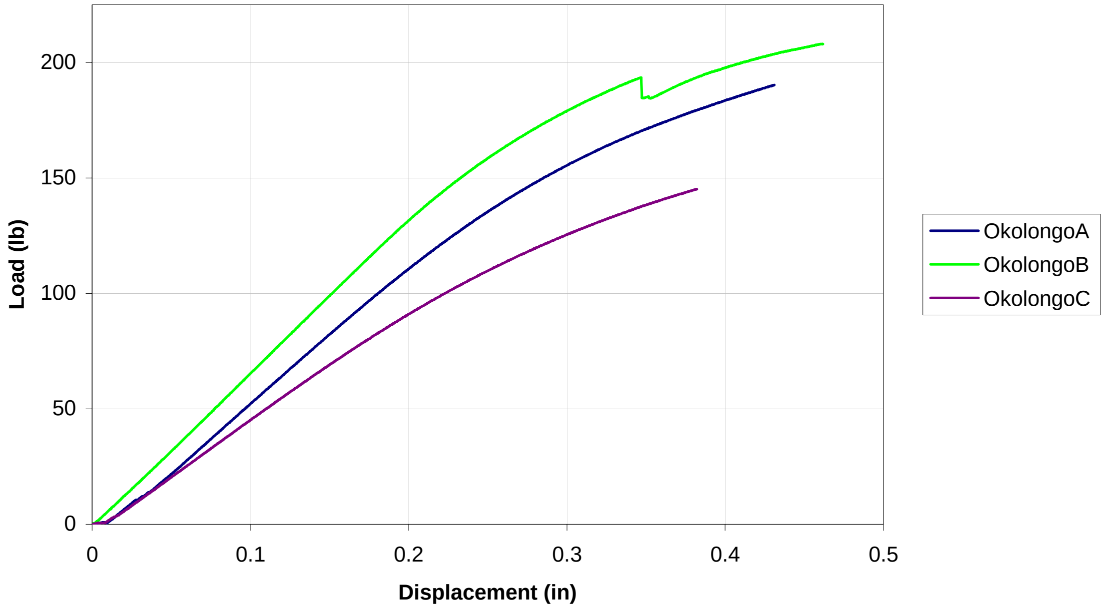




	k (lb/in)	E (10^3 psi)	F_{max} (lb)	σ_{max} (psi)
DihakeA	435.71	1267.5	128.77	6280
DihakeB	398.49	1159.2	107.62	5249
DihakeC	347.51	1010.9	103.65	5055
Average		1145.9		5528

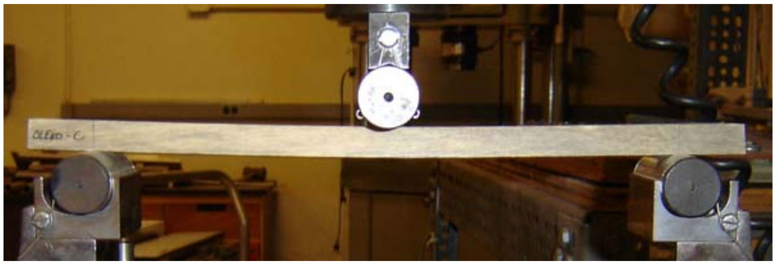
Okolongo

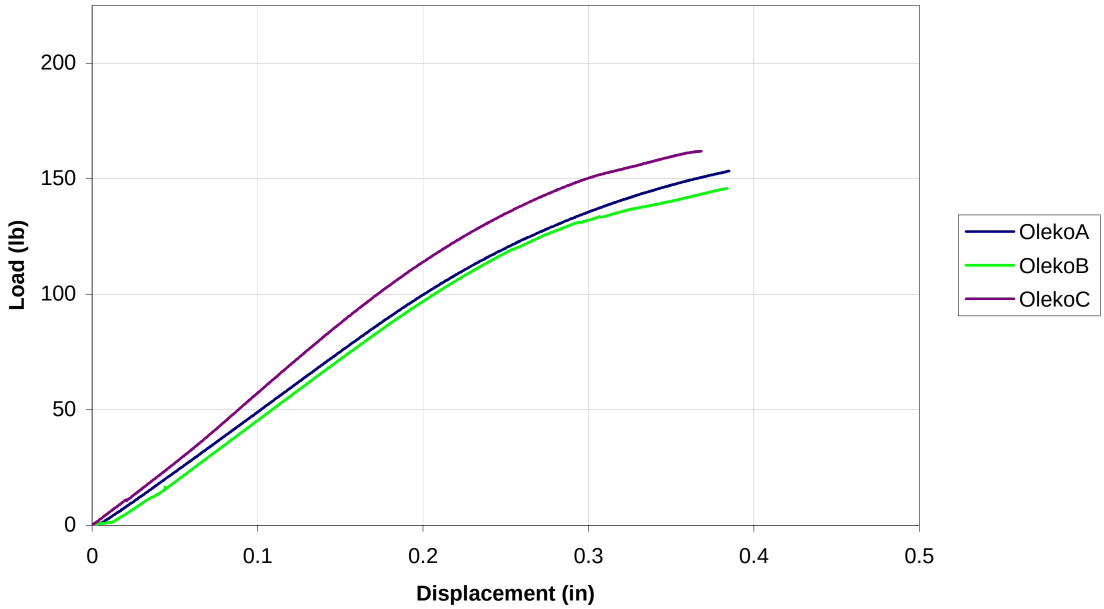




	k (lb/in)	E (10^3 psi)	F_{max} (lb)	σ_{max} (psi)
OkolongoA	608.62	1770.5	190.23	9278
OkolongoB	673.17	1958.3	208.01	10145
OkolongoC	496.16	1443.4	145.12	7078
Average		1724.1		8833

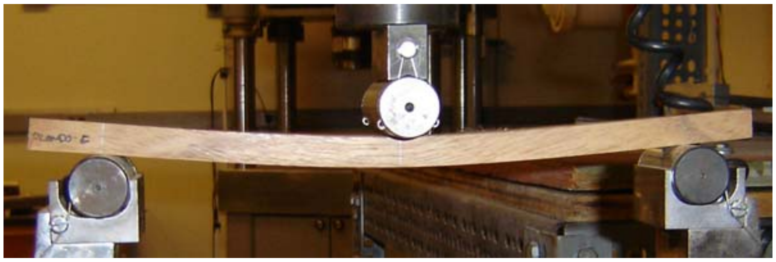
Oleko

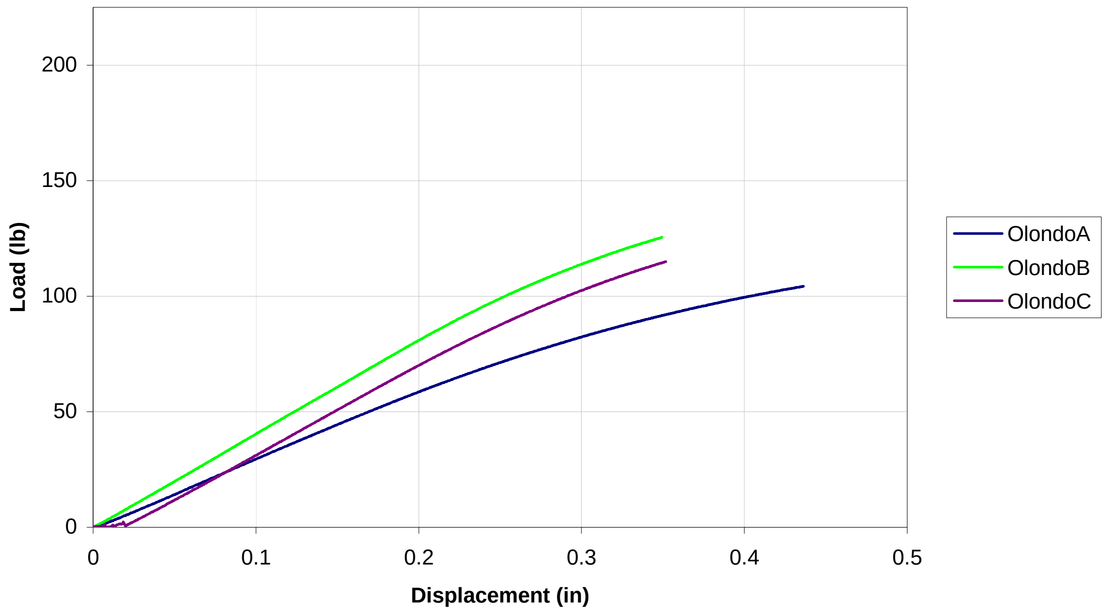




	k (lb/in)	E (10^3 psi)	F_{max} (lb)	σ_{max} (psi)
OlekoA	519.46	1511.2	153.31	7477
OlekoB	532.77	1549.9	145.87	7114
OlekoC	603.77	1756.4	161.92	7897
Average		1605.8		7496

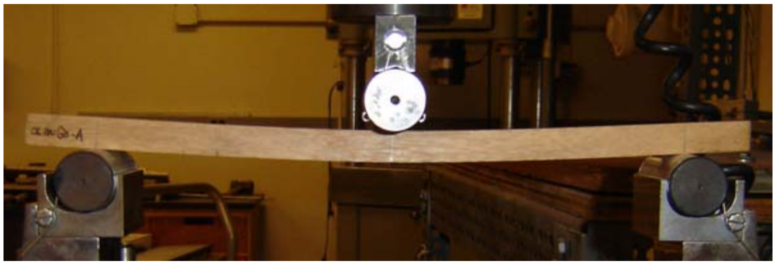
Olondo

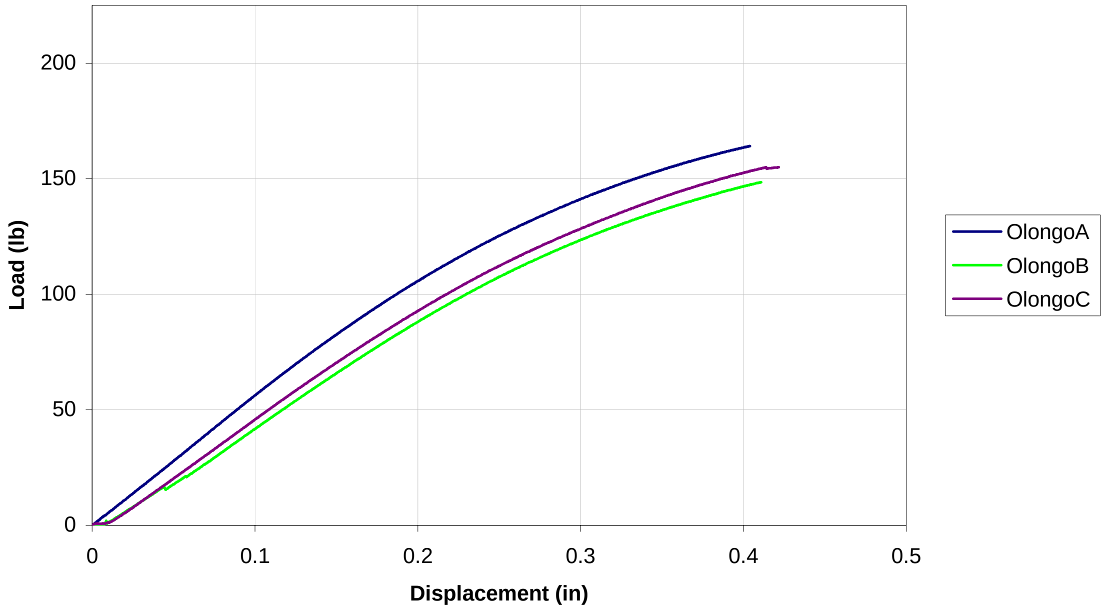




	k (lb/in)	E (10^3 psi)	F_{max} (lb)	σ_{max} (psi)
OlondoA	307.17	893.6	104.30	5087
OlondoB	411.04	1195.8	125.46	6119
OlondoC	388.06	1128.9	114.95	5606
Average		1072.8		5604

Olongo





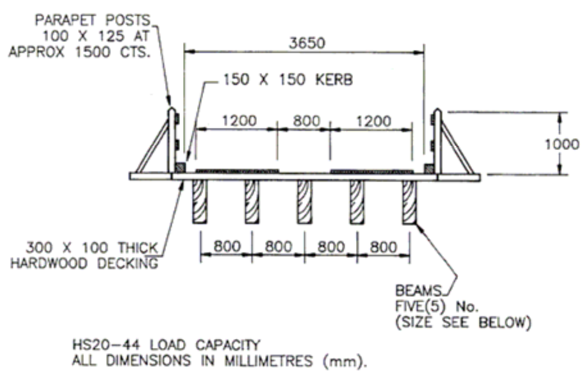
	k (lb/in)	E (10^3 psi)	F_{max} (lb)	σ_{max} (psi)
OlongoA	568.05	1652.5	164.11	8004
OlongoB	478.43	1391.8	148.43	7239
OlongoC	510.26	1484.4	154.97	7558
Average		1509.6		7600

Appendix B - Calculations for Timber Bridges

This appendix provides detailed calculations and results for three existing timber bridge systems intended for developing countries: a timber beam bridge, the Allotey built-up timber girder, and the Kenyan Low-Cost Modular Timber Bridge.

B.1 - Timber Beam Bridge

A standard single-lane sawn timber beam bridge is shown in Figure B.1. Design properties are based on AASHTO HS20-44 loading and are given for a variety of spans and for three wood species groups, heavy hardwood, lighter hardwood, and softwood. Design values for these groups are given in Table B.1. Heavy hardwoods are defined as having a specific gravity (SG) greater than 0.65 at 18% moisture content (MC), lighter hardwoods are defined as hardwoods having an SG less than 0.65 at 18% MC, and softwoods are a subset of softwoods with an SG greater than 0.42 at 18% MC, which are those considered suitable for bridge construction (Transport Research Laboratory 2000).



SPAN (m)	BEAM SIZE FOR TIMBER GROUP		
	GROUP A	GROUP B	GROUP C
4	150 X 375	150 X 500	200 X 550
6	150 X 475	200 X 550	200 X 700
8	200 X 500	200 X 650	250 X 750
10	200 X 600	250 X 725	300 X 850
12	200 X 700	250 X 850	300 X 1000

Figure B.1 - Standard single-lane sawn timber beam bridge (Transport Research Laboratory 2000)

Table B.1 - Permissible short-term stresses for wood groups (Transport Research Laboratory 2000)

Design Values: MPa (ksi)	Group A: Heavy Hardwoods	Group B: Lighter Hardwoods	Group C: Softwoods
Bending	15.1 (2.19)	8.6 (1.25)	5.4 (0.78)
Tension	9.0 (1.31)	5.0 (0.73)	3.2 (0.46)
Compression parallel to the grain	11.3 (1.64)	6.8 (0.98)	5.0 (0.73)
Compression perpendicular to the grain	2.2 (0.32)	1.8 (0.26)	1.5 (0.22)
Shear parallel to the grain	2.2 (0.32)	1.1 (0.16)	0.9 (0.13)

Live load design moments and end shears are taken directly from AASHTO HS20-44 design tables for the given spans, and are presented with the analysis results in Table B.2. Dead loads must be determined based on the weight of the material. Density, ρ, is taken as the maximum value for the wood group, 650 kg/m^3 for group B. This is converted into a unit weight by multiplying by the acceleration of gravity. A uniform distributed line load is then derived by multiplying unit weight by the overall cross-sectional area. The cross-sectional area can be estimated as the sum of the cross sections of the beams, deck, running boards, and curb. Railings supports and posts are ignored, as they are discontinuous, but this is approximately balanced by ignoring gaps between deck planks. Running boards are taken to have a thickness of 50 mm, equal to half the thickness of the deck planks.

$$A_{tot} = A_{beams} + A_{deck} + A_{boards} + A_{curb}$$
$$A_{tot} = 5 \cdot b \cdot h + 0.1 \cdot (3.65 + 2 \cdot 0.15) + 2 \cdot 0.05 \cdot 1.2 + 2 \cdot 0.15 \cdot 0.15 \quad (\text{m}^2)$$
$$A_{tot} = 5 \cdot b \cdot h + 0.56 \ (\text{m}^2)$$

where b and h and the beam thickness and height, respectively.

This yields a uniformly distributed dead load of:

$$w_{DL} = 9.81 \cdot \rho \cdot A_{tot}$$
$$w_{DL} = 9.81 \cdot 650 \cdot (5 \cdot b \cdot h + 0.56) \quad (\text{N/m})$$

Maximum moment and shear for a uniformly distributed load on a simply-supported beam are calculated as:

$$M_{\max,DL} = \frac{w_{DL} \cdot L^2}{8} \text{ and } S_{\max,DL} = \frac{w_{DL} \cdot L}{2}$$

where L is the span. These results are combined with the moment and shear from the live load, M_{LL} and S_{LL}, to find the total design moment and end shear. Finally, the maximum bending stress and shear stress can be determined from standard formulae for rectangular beams. Since there are five beams, the applied bending moment and shear are divided by five to determine the stress in a given beam.

$$f_b = \frac{6 \cdot M}{b \cdot d^2} = \frac{6 \cdot (M_{LL} + M_{DL})}{5 \cdot b \cdot d^2}$$

and

$$f_v = \frac{3}{2}\frac{S}{b \cdot d} = \frac{3}{10}\frac{(S_{LL} + S_{DL})}{b \cdot d}$$

These values must be compared with the allowable bending stress, F_b, and the allowable shear stress, F_v.

Results of design calculations are presented in Table B.2. The resulting ratio of applied bending stress to permissible bending stress has an average value of approximately 0.59 with a minimum ratio of 0.57 and a maximum ratio of 0.60. These ratios are relatively constant, both within and across wood groups, and are assumed to be the primary design criteria. It is not clear why there is such a significant discrepancy between maximum design stress and permissible stress. It may account for strength reduction factors (such as the wet service factor), live load distribution factors, or may simply include a further factor of safety beyond those intrinsic to the permissible stress.

Table B.2 - Summary of design results for Group B timber beam bridge supporting HS20-44 loading

			ρ	650	kg/m^3						
	Design Values		F_b	8.6	Mpa						
	HS20-44		F_v	1.1	MPa						
Span (m)	M_{LL} (kN-m)	S_{LL} (kN)	b (mm)	h (mm)	w_{DL} (kN/m)	M_{DL} (kN-m)	S_{DL} (kN-m)	f_b (Mpa)	f_v (Mpa)	f_b/F_b	f_v/F_v
4	141	142.3	150	500	5.96	11.92	11.92	4.89	0.62	0.57	0.56
6	217	185	200	550	7.08	31.85	21.23	4.94	0.56	0.57	0.51
8	301.3	216.2	200	650	7.72	61.72	30.86	5.16	0.57	0.60	0.52
10	444.4	229.5	250	725	9.35	116.87	46.75	5.13	0.46	0.60	0.42
12	585.9	243.8	250	850	10.35	186.23	62.08	5.13	0.43	0.60	0.39

The ratio of permissible shear stress to applied shear stress is highly variable, ranging from a maximum of 0.56 to a minimum of 0.39 for group B, and lower for other wood groups. The size of these ratios implies that shear stress is of secondary concern to bending stress in these types of structures. The applied shear stress given in the tables is also somewhat overestimated. When designing for shear in wooden bending members, it is allowable to reduce or ignore loads applied "within a distance from supports equal to the depth of the bending member" (AF&PA 2005). This would reduce the applied end shear from both the live and dead loads, and have a corresponding effect on the applied shear stress. For the purposes of this simple analysis, this reduction is ignored.

Having determined the design criteria, the design can be modified to account for the lower load levels defined by an H10-44 loading. Results are shown in Table B.3. Similar ratios of permissible to applied bending stress were maintained, as can be seen in the similarity of ratios between the designs.

Table B.3 - Summary of redesign results for Group B timber beam bridge to support H10-44 loading

Design Values H10-44			
	ρ	650	kg/m^3
	F_b	8.6	Mpa
	F_v	1.1	MPa

Span (m)	M_{LL} (kN-m)	S_{LL} (kN)	b (mm)	h (mm)	w_{DL} (kN/m)	M_{DL} (kN-m)	S_{DL} (kN-m)	f_b (Mpa)	f_v (Mpa)	f_b/F_b	f_v/F_v
4	70.5	71.2	150	350	5.24	10.49	10.49	5.29	0.47	0.62	0.42
6	80	76.5	150	400	5.48	24.68	16.45	5.23	0.46	0.61	0.42
8	104	79.2	200	425	6.28	50.25	25.12	5.12	0.37	0.60	0.33
10	138.9	81.4	250	475	7.36	91.96	36.78	4.91	0.30	0.57	0.27
12	193.9	88.5	250	575	8.15	146.77	48.92	4.95	0.29	0.58	0.26

B.2 - Allotey Built-up Timber Girder

The Allotey built-up timber girder, to be referred to henceforth as the "Allotey girder", consists of a web made up of two layers of diagonal members, one layer the mirror of the other, with two longitudinal side flange members attached at the top and bottom as well as optional top and bottom flange members. The girder is entirely fabricated with mechanical fasteners, with bolts at all diagonal intersections, and nails connecting the top and bottom flanges to the side flanges. Schematic views of the truss are shown in Figure B.2.

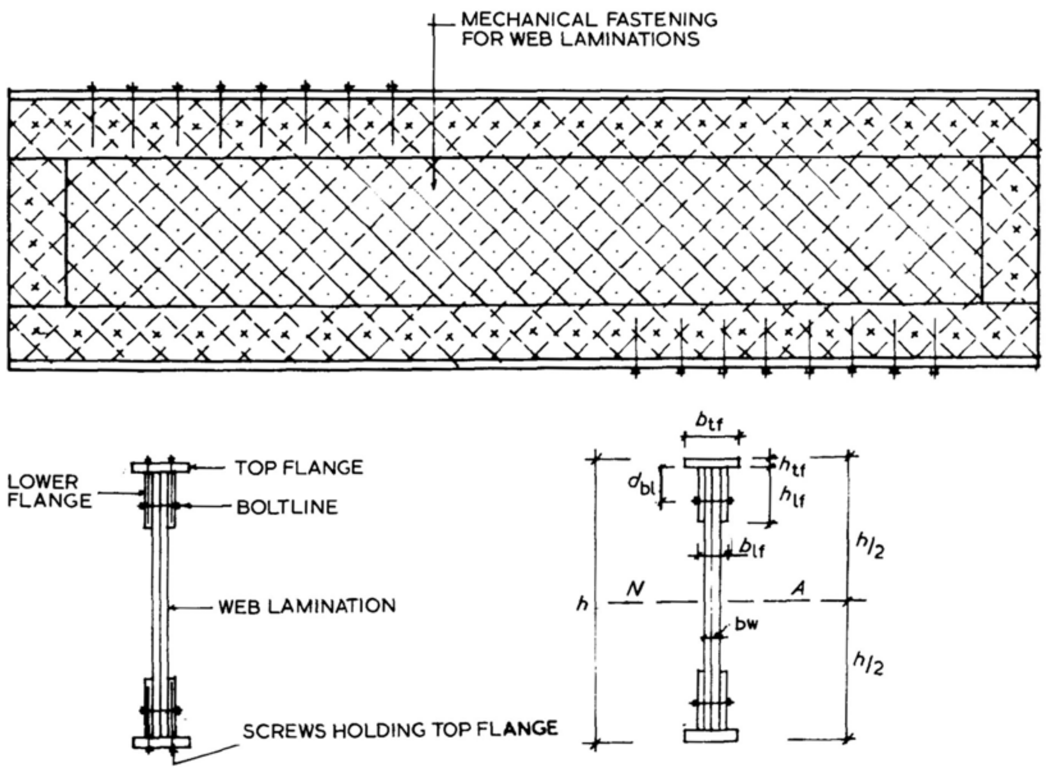


Figure B.2 - Schematic views and notation for Allotey Built-up Timber Girder (Allotey 1988)

Assuming the same timber is used for both web and flange members, the maximum axial stress in the flange is given to be:

$$f_{f,m} = \frac{M \cdot h}{2 \cdot I_e}$$

where I_e is an effective moment of inertia, calculated as:

$$I_e = I_w \cos^4 \mu + I_f$$

where I_w and I_f are the moments of inertia of the web and flange, respectively, with respect to the centre of the overall section. For the given cross-section, these values can be calculated as:

$$I_e = I_{w,c} \cdot \cos^4 \mu + I_{lf,c} + I_{lf,pa} + I_{tf,c} + I_{tf,pa}$$

$$I_e = \frac{1}{12} \cdot (b_w) \cdot (h - 2 \cdot h_{tf} - d_{bl})^3 \cdot \cos^4 \mu$$

$$+ 2 \cdot \frac{1}{12} \cdot (b_{lf} - b_w) \cdot (h_{lf})^3 + 2 \cdot (b_{lf} - b_w) \cdot (h_{lf}) \cdot \left(\frac{h}{2} - h_{tf} - \frac{h_{lf}}{2}\right)^2$$

$$+ 2 \cdot \frac{1}{12} \cdot (b_{tf}) \cdot (h_{tf})^3 + 2 \cdot (b_{tf}) \cdot (h_{tf}) \cdot \left(\frac{h}{2} - \frac{h_{tf}}{2}\right)^2$$

The calculation of moment of inertia for the built-up section is illustrated in Figure B.3.

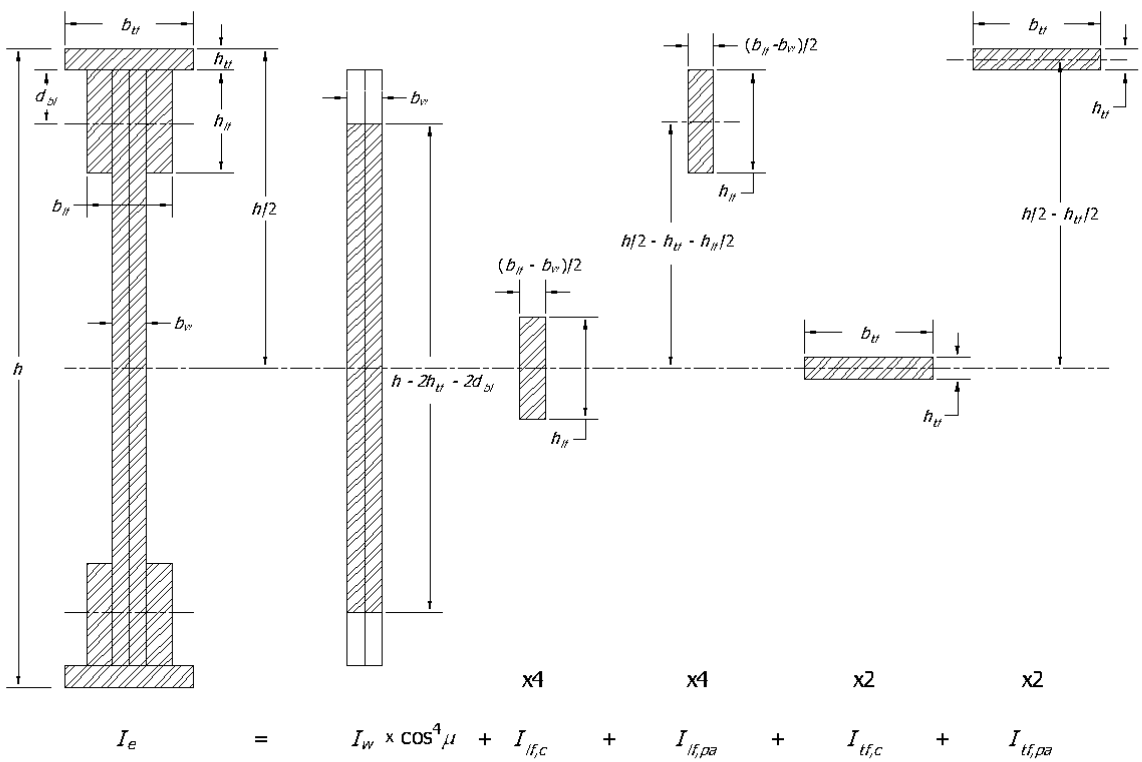


Figure B.3 - Schematic representation of moment of inertia calculation for Allotey girder

Shear transfer in the beam is carried as axial force in the diagonals. The maximum axial force due to shear can be calculated as:

$$f_{w,s} = \frac{\pm 2 \cdot S \cdot Q_e}{b_w \cdot I_e \cdot \sin 2\mu}$$

where Q_e is an effective static moment of area, calculated as:

$$Q_e = \cos^4 \mu \cdot Q_w + Q_f$$

where Q_f is the static moment of area of either the upper or lower flange members and Q_w is the static moment of area of the loaded potion of either the top or bottom half of the web (i.e. the area within the outermost bolt lines). These can be combined, for the given cross-section, to yield:

$$Q_e = \cos^4 \mu \cdot Q_w + Q_{lf} + Q_{tf}$$

$$Q_e = +\cos^4 \mu \cdot \left(b_w \cdot \left(\frac{h}{2} - h_{tf} - d_{bl} \right) \cdot \frac{1}{2} \cdot \left(\frac{h}{2} - h_{tf} - d_{bl} \right) \right)$$
$$+ \left(b_{lf} - b_w \right) \cdot h_{lf} \cdot \left(\frac{h}{2} - h_{tf} - \frac{h_{tf}}{2} \right) + b_{tf} \cdot h_{tf} \cdot \left(\frac{h}{2} - \frac{h_{tf}}{2} \right)$$

This calculation for the static moment of area for the built-up section is illustrated in Figure B.4.

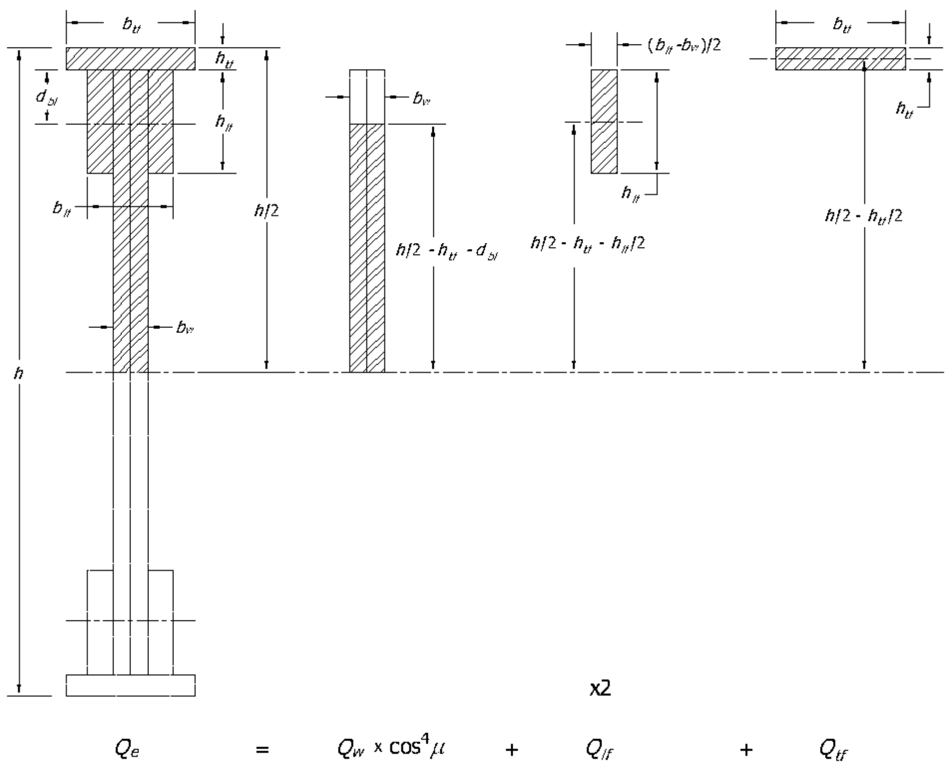


Figure B.4 - Schematic representation of static moment of area calculation for Allotey girder

A design was performed to determine an appropriate design for a bridge to support H10-44 loading. An example rural road bridge cross-section would have two girders spaced at 2.1 m (7 ft) on-centre, with a transverse 102mm (4 in) deep nail nail-laminated deck. The girder was designed to meet the permissible stresses for a Group B wood as given in Table B.1. The resulting properties for a span of 18.3 m (60 ft), the maximum soan suggested by Allotey, are:

Overall:	$h = 1820$ mm	$\mu = 45°$ (angle of web diagonals from horizontal)
Lower flange:	$b_{lf} = 254$ mm	$h_{lf} = 305$ mm
Web:	$b_w = 102$ mm	
Bolt line:	$d_{bl} = 115$ mm	

The top flanges are not included in the design. Allotey's prototype included top flanges for lateral resistance during testing, but did not include them in the initial determination of dimensions based on moment capacity.

Density, ρ, is taken as the maximum value for the wood group, 650 kg/m^3 for group B. A uniform distributed line load is then derived by converting density to unit weight and multiplying it by the overall cross-sectional area. The cross-sectional area can be estimated as the sum of the cross sections of the girders and the deck.

$$A_{tot} = 2 \cdot A_{girder} + A_{deck}$$
$$A_{tot} = 2 \cdot \left(2 \cdot b_{tf} \cdot h_{tf} + 2 \cdot (b_{lf} - b_w) \cdot h_{lf} + b_w \cdot (h - 2 \cdot h_{tf})\right) + d \cdot w$$
$$A_{tot} = 2 \cdot (2 \cdot 0 \cdot 0 + 2 \cdot (254 - 102) \cdot 305 + 102 \cdot (1905 - 2 \cdot 0)) + 102 \cdot 3660$$
$$A_{tot} = 2 \cdot \left(0 + 9.27 \cdot 10^4 + 1.85 \cdot 10^5\right) + 3.73 \cdot 10^5 = 9.47 \cdot 10^5 \text{ mm}^2$$

This yields a uniformly distributed dead load of:

$$w_{DL} = 9.81 \cdot \rho \cdot A_{tot}$$
$$w_{DL} = 9.81 \cdot 650 \cdot \left(\frac{9.28 \cdot 10^5}{1000^2}\right) = 5.92 \cdot 10^3 \text{ N/m}$$

Maximum moment and shear for a uniformly distributed load on a simply-supported beam are calculated as:

$$M_{\max,DL} = \frac{w_{DL} \cdot L^2}{8} = \frac{5.92 \cdot 10^3 \cdot 18.3^2}{8} = 247 \cdot 10^3 \text{ N-m}$$
$$S_{\max,DL} = \frac{w_{DL} \cdot L}{2} = \frac{5.92 \cdot 10^3 \cdot 18.3}{2} = 54 \cdot 10^3 \text{ N}$$

Assuming load is supported evenly by the two girders, this leads to final maximum axial stress in the flange due to moment and maximum axial stress in the web due to shear.

$$f_{f,m} = \frac{M \cdot h}{2 \cdot I_e} = \frac{1000 \cdot \left(247 \cdot 10^3 + 622 \cdot 10^3\right) / 2 \cdot 1810}{2 \cdot \left(1.03 \cdot 10^{10} + 7.19 \cdot 10^8 + 5.25 \cdot 10^{10} + 0 + 0\right)} = 6.19 \text{ MPa}$$

$$f_{w,s} = \frac{\pm 2 \cdot S}{b_w \cdot I_e \cdot \sin 2\mu}\left(\cos^4 \mu \cdot Q_w + Q_f\right)$$
$$= \frac{\pm 2 \cdot \left(54 \cdot 10^3 + 136 \cdot 10^3\right)/2}{102 \cdot 6.36 \cdot 10^{10} \cdot \sin(2 \cdot 45)} \cdot \left(7.96 \cdot 10^6 + 3.49 \cdot 10^7 + 0\right) = \pm 1.26 \text{ MPa}$$

Both values are below the adjusted permissible value for stress in tension parallel to grain of 6.25 MPa, calculated using the base permissible stress from Table B.1 and a load duration adjustment factor, C_d, of 1.25.

Example design results for timber bridges using the Allotey girder system with 2 girders are shown in Table B.4. The analysis demonstrated above was used iteratively to develop adequate designs for spans ranging from 4 to 18.3 m.

Table B.4 - Design results for Allotey Built-up Girder for spans from 4 to 18.3 m

Geometry									
Span	m	4.0	6.0	8.0	10.0	12.0	14.0	16.0	18.3
h	mm	640	760	805	900	1060	1240	1325	1810
μ	degs	45	45	45	45	45	45	45	45
b_{lf}	mm	203	203	229	229	229	229	254	254
h_{lf}	mm	178	203	203	254	254	279	279	305
b_w	mm	102	102	102	102	102	102	102	102
d_{bl}	mm	131	128	151	127	126	135	123	115
I_e	mm^4	2.29E+09	3.86E+09	5.44E+09	8.06E+09	1.26E+10	1.97E+10	2.74E+10	6.36E+10
Q_e	mm^3	4.61E+06	6.52E+06	8.57E+06	1.17E+07	1.51E+07	2.00E+07	2.59E+07	4.28E+07
Dead Load									
ρ	kg/m^3	650	650	650	650	650	650	650	650
Girder									
A	mm^2	101236	118526	133672	156316	172636	197346	219966	277340
2*A	mm^2	202472	237052	267344	312632	345272	394692	439932	554680
w_{girder}	N/m/girder	645.53	755.78	852.36	996.75	1100.81	1258.38	1402.61	1768.46
Decking									
d	mm	102	102	102	102	102	102	102	102
w	m	3.66	3.66	3.66	3.66	3.66	3.66	3.66	3.66
w_{deck}	N/m	2371.1	2371.1	2371.1	2371.1	2371.1	2371.1	2371.1	2371.1
Design Loads									
M_{DL}	kN-m	7	17	33	55	82	120	166	247
S_{DL}	kN	7	12	16	22	27	34	41	54
M_{LL}	kN-m	81	108	136	167	212	277	350	622
S_{LL}	kN	73	77	79	81	83	91	102	136
Maximum Stresses									
$f_{f,m}$	MPa	6.19	6.20	6.22	6.19	6.21	6.24	6.23	6.19
$f_{w,s}$	MPa	1.57	1.46	1.47	1.46	1.30	1.25	1.33	1.26
Permissible Stresses Group B (x1.25 for Cd)									
F'_t	MPa	6.25	6.25	6.25	6.25	6.25	6.25	6.25	6.25
F'_c	MPa	8.50	8.50	8.50	8.50	8.50	8.50	8.50	8.50

B.3 - Kenyan Low-Cost Modular Timber Bridge

A type of modular timber truss was developed in the Forest Department of the Ministry of Natural Resources in Kenya for use in carrying light trucks in rural areas (Parry 1981). Dubbed the Kenyan Low-Cost Modular Timber Bridge, the system was developed, published, and has been used in a significant number of bridges in Kenyan and other countries in Africa.

The truss is constructed from a series of modular inverted king post trusses, a schematic of which is shown in Figure B.5. Trusses are connected together along the top compression chords using a system of male/female end plates with shear pins. Steel bottom chords are used to connect the points of the triangular sections together and support the tension forces. The suggested number of trusses needed for different spans and loadings are given in Table B.5.

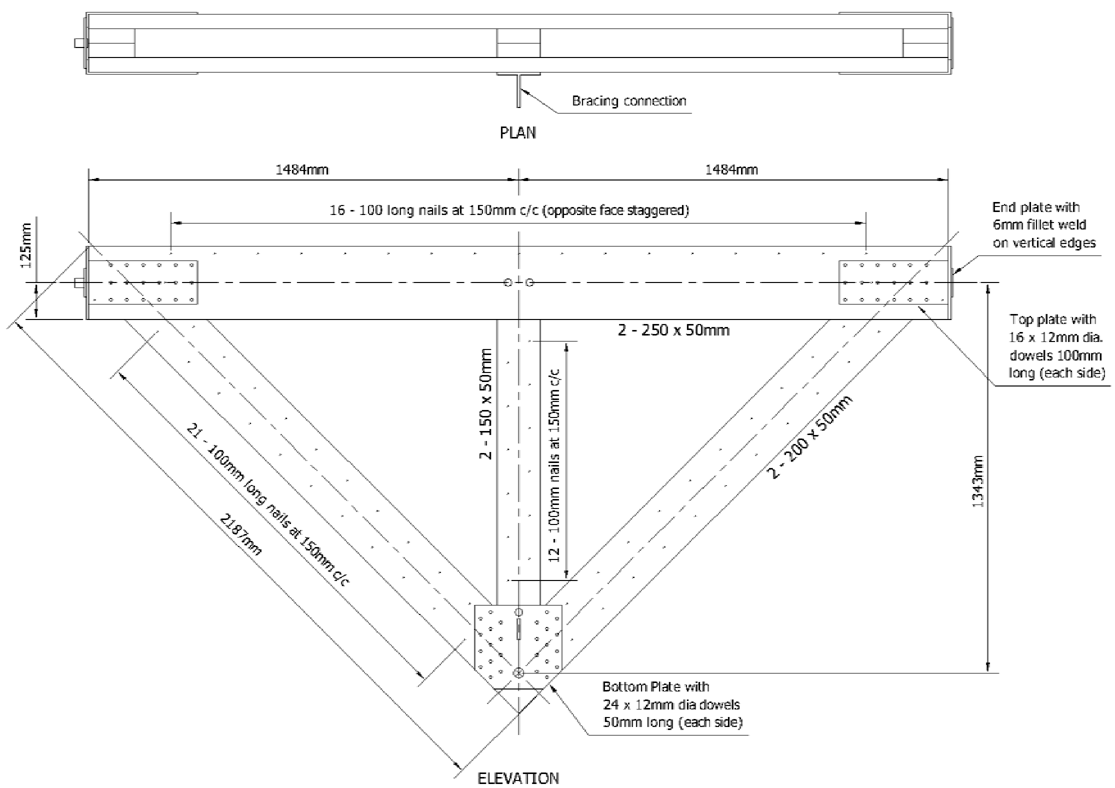


Figure B.5 - Schematic of single module of Kenyan Low-Cost Modular Timber Bridge (Parry 1981)

Table B.5 - Number of trusses required in Kenyan Low-Cost Modular Timber Bridge for various spans and loadings (Parry 1981)

Loading duty	Span					
	12m	15m	18m	21m	24m	27m
HA*	6	8	–	–	–	–
H20–44*	4	4	6	6	8	–
H10–44*	2	2	4	4	4	6

For an H10-44 loading, 2 trusses are suggested to be sufficient for spans up to at least 15m (~50 ft), as shown in Table B.5. Each truss would be constructed from five three-meter-long modules. It is not clear if the trusses were originally designed to support a full pedestrian loading, which has been shown to dominate for both shear and bending. Therefore, loading will be performed for pedestrian loading separately from vehicle loading to determine dominant loading for individual members.

For a two-truss bridge, the cross-section will match that shown in Figure B.6.

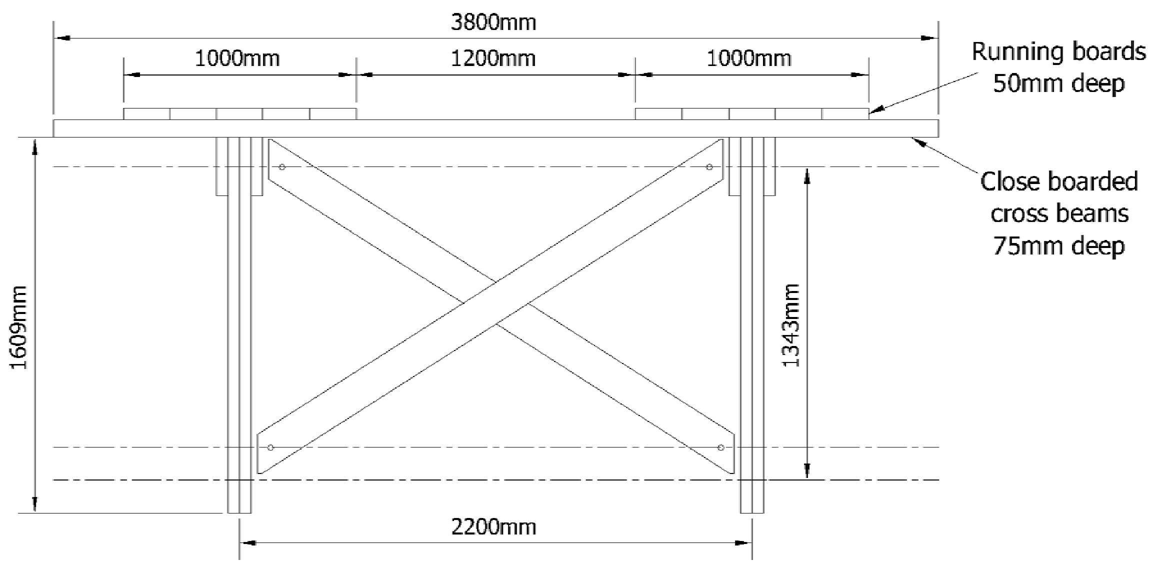


Figure B.6 - Cross-section of 2-truss Kenyan Truss bridge

Individual truss modules are given to weigh approximately 140 kg each. This value is scaled up by the weight of two additional 50 x 300mm top chord members which are added following recommendations by Parry. This modification is illustrated in Figure B.7.

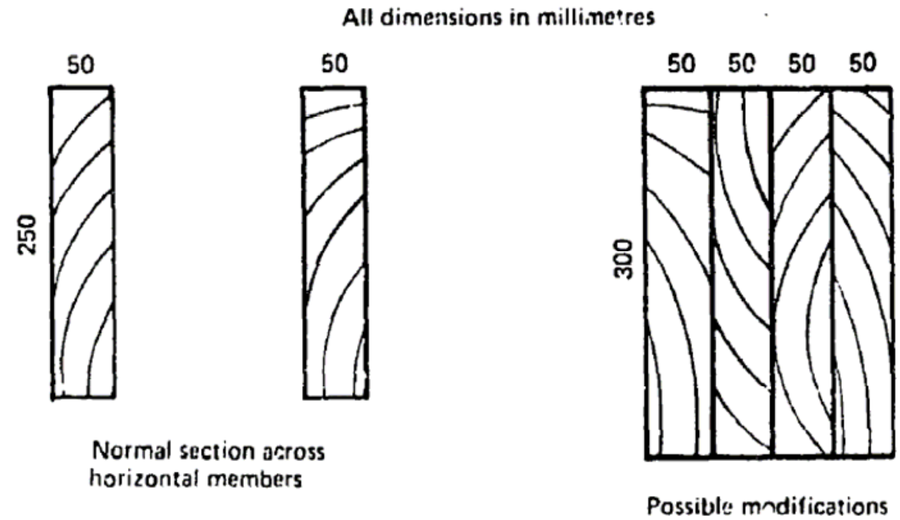


Figure B.7 - Recommended Kenyan truss top chord modification (Parry 1981)

Module weights can be converted into an equivalent line load by multiplying by the number of lines of trusses and dividing by the length of a single truss. A density of 650 kg/m^3 is assumed for all timber in the bridge.

$$w_{DL,truss} = \frac{2 \cdot 9.81 \cdot (140 + 650 \cdot 2 \cdot 0.05 \cdot 0.3 \cdot 3.0)}{3} = 1298.2 \text{ N/m}$$

Since the deck and running boards are continuous over the length of the bridge, they can be converted into a line load by multiplying the cross-sectional area by the unit weight of the material. A density of 650 kg/m3 is assumed for timber.

$$w_{DL,deck} = 9.81 \cdot 650 \cdot (0.075 \cdot 3.8 + 2 \cdot 0.05 \cdot 1.0) = 2455 \text{ N/m}$$

Vertical cross-bracing must be provided for each module. Thus, the weight of a pair of cross-braces can be divided over the module length to yield an equivalent line load. Exact dimensions for cross-bracing are provided for bridges with 4 or more trusses, which are paired with a much shorter spacing. Therefore the length of the longer cross-bracing for two trusses must be estimated, and is taken as 150mm less than the diagonal of the centre-line distances shown in Figure B.6.

$$w_{DL,CB} = 9.81 \cdot 650 \cdot 2 \cdot \frac{0.15 \cdot 0.05 \cdot \left(\sqrt{2.2^2 + 1.343^2} - 0.15\right)}{3} = 77.5 \text{ N/m}$$

This loading will be doubled to account for the unknown parameters of the horizontal cross-bracing.

Finally, the weight of the steel lower-chords can be included. A schematic of a single plate is shown in Figure B.8. Each truss has two continuous strings of members, one attached on each side as shown in Figure B.9.

$$w_{CB,steel} = 9.81 \cdot 7860 \cdot 4 \cdot \frac{3.1 \cdot 0.1 \cdot 0.006 + 2 \cdot 0.1 \cdot 0.075 \cdot 0.006 - 2 \cdot \pi \cdot 0.02^2 \cdot 0.012}{3}$$

$$= 197 \text{ N/m}$$

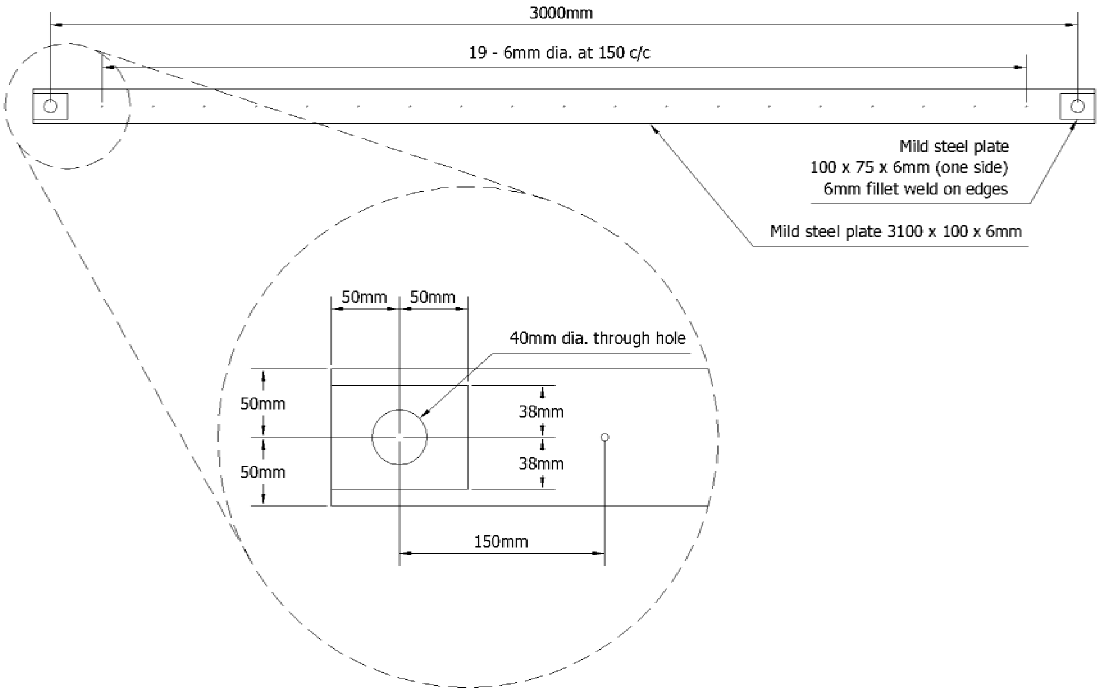


Figure B.8 –Schematic of steel bottom chord member from Kenyan Truss (Reproduced from Parry 1981)

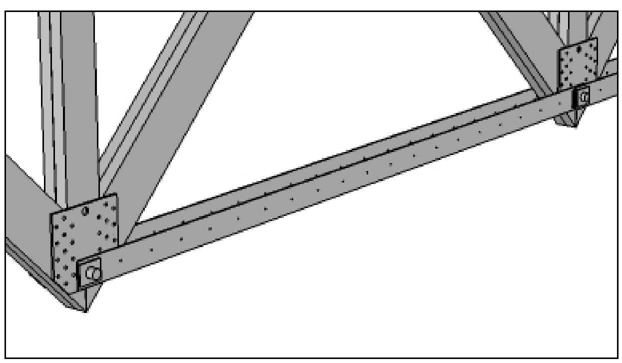


Figure B.9 - Model of bottom chord structure in Kenyan Truss

Combining the line loads above yields a final total distributed dead load of:

$$w_{DL} = w_{DL,truss} + w_{DL,deck} + 2 \cdot w_{DL,CB} + w_{DL,steel} = 1043 + 2455 + 155 + 197 = 4105 \text{ N/m}$$

Live loading will be based on the greater of lane loading or a design truck for each member of the truss. Both types of loading include point loads, which must be run across the length of the bridge to determine the worst case loading. In order to have resolution finer than the spacing of the nodes, an algorithm for distributing the force to nodes must be established. Distributed line loads will be split between truss nodes based on simple tributary area methods.

The distribution of point loads to nodal forces is determined by treating the top chord of the truss as a continuous beam with three supports as shown in Figure B.10.

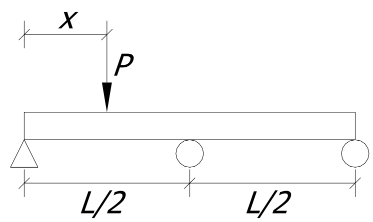


Figure B.10 - Beam model of top chord of Kenyan Truss module

We can use geometric compatibility at mid-span to solve for the centre reaction (R_C), which will represent load applied at the centre node of the truss top chord. For $x \leq L/2$, the downwards deflection at mid-span due to load P applied at arbitrary point x is:

$$\Delta_1 = \frac{P}{12EI} \cdot \left(\frac{3}{4} L^2 x - x^3 \right)$$

The upwards deflection due to reaction R_C applied at mid-span is:

$$\Delta_2 = \frac{R_c L^3}{48EI}$$

The sum of the two deflections must be equal to zero to satisfy the requirements of the mid-span support, meaning the two deflections must be equal.

$$\Delta_2 = \Delta_1$$

$$\frac{R_c L^3}{48EI} = \frac{P}{12EI} \cdot \left(\frac{3}{4} L^2 x - x^3 \right)$$

$$R_c = P \cdot \left(3\frac{x}{L} - 4\frac{x^3}{L^3} \right)$$

The reactions at the left and right supports (R_L and R_R), which will represent the applied load at each node respectively, can be calculated as the sum of the resulting reactions for the point load P applied on a simply supported beam and the resulting reactions for the centre reaction R_C applied on a simply-supported beam.

$$R_L = P \cdot \left(1 - \frac{x}{L} \right) - \frac{P}{2} \cdot \left(3\frac{x}{L} - 4\frac{x^3}{L^3} \right) = P \cdot \left(1 - \frac{5}{2}\frac{x}{L} + 2\frac{x^3}{L^3} \right)$$

$$R_R = P \cdot \left(\frac{x}{L} \right) - \frac{P}{2} \cdot \left(3\frac{x}{L} - 4\frac{x^3}{L^3} \right) = P \cdot \left(-\frac{1}{2}\frac{x}{L} + 2\frac{x^3}{L^3} \right)$$

A similar procedure can be used for $x \geq L/2$. In this case, the downwards deflection at mid-span due to load P applied at arbitrary point x is:

$$\Delta_1 = \frac{P}{12EI} \cdot \left(-\frac{1}{4}L^3 + \frac{9}{4}L^2 x - 3Lx^2 + x^3 \right)$$

Equating deflections yields:

$$\frac{R_c L^3}{48EI} = \frac{P}{12EI} \cdot \left(-\frac{1}{4}L^3 + \frac{9}{4}L^2 x - 3Lx^2 + x^3 \right)$$

$$R_c = P \cdot \left(-1 + 9\frac{x}{L} - 12\frac{x^2}{L^2} + 4\frac{x^3}{L^3} \right)$$

Again, the reactions from the two simply-supported beams can be combined to find the reactions on the full beam.

$$R_L = P \cdot \left(1 - \frac{x}{L} \right) - \frac{P}{2} \cdot \left(-1 + 9\frac{x}{L} - 12\frac{x^2}{L^2} + 4\frac{x^3}{L^3} \right)$$

$$= P \cdot \left(\frac{3}{2} - \frac{11}{2}\frac{x}{L} + 6\frac{x^2}{L^2} - 2\frac{x^3}{L^3} \right)$$

$$R_R = P \cdot \left(\frac{x}{L} \right) - \frac{P}{2} \cdot \left(-1 + 9\frac{x}{L} - 12\frac{x^2}{L^2} + 4\frac{x^3}{L^3} \right)$$

$$= P \cdot \left(\frac{1}{2} - \frac{7}{2}\frac{x}{L} + 6\frac{x^2}{L^2} - 2\frac{x^3}{L^3} \right)$$

The reactions of the two-span beam can be used to determine the internal moments. The maximum moment is taken to occur under the load when it is located at a module quarter point, or $x = L/4$ in Figure B.10.

$$M_{\max}\left(x = \frac{L}{4} \right) = R_L\left(x = \frac{L}{4} \right) \cdot \frac{L}{4} = P \cdot \left(1 - \frac{5}{2}\frac{L/4}{L} + 2\frac{(L/4)^3}{L^3} \right) \cdot \frac{L}{4}$$

$$= P \cdot \left(\frac{L}{4} - \frac{5}{32}L + \frac{1}{128}L \right) = \frac{13}{128}PL$$

AASHTO H10-44 loading requires a maximum axle-load of 71 kN, which must be divided between the supporting trusses. For a two-truss bridge, the load will be divided evenly, yielding an applied load, *P*, of 35.5 kN. For a module length, *L*, of 3m, the maximum moment will be 10.8 kN-m.

Applied bending moment can be converted into maximum bending stress using the standard formula for a rectangular section. For a two-truss bridge, this yields:

$$f_b = \frac{6 \cdot M}{b \cdot d^2} = \frac{6 \cdot (10.82 \cdot 1000 \cdot 1000)}{(4 \cdot 50) \cdot 300^2} = 3.61 \text{ MPa}$$

Pedestrian loading will also induce bending in the top chord of the truss. Maximum moment will again occur at the module quarter point, and can be conservatively calculated assuming the top chord of each module is made up of two simply-supported beams of length $L/2$. The maximum moment for a two truss bridge (i.e. N_{truss} = 2) can therefore be calculated as:

$$M_{\max} = \frac{w_{ped} \cdot width_{deck} \cdot (L/2)^2}{8 \cdot N_{truss}} = \frac{4 \cdot 3.8 \cdot (1.5)^2}{8 \cdot 2} = 2.14 \text{ kN-m}$$

which yields a maximum bending stress of:

$$f_b = \frac{6 \cdot M}{b \cdot d^2} = \frac{6 \cdot (2.14 \cdot 1000 \cdot 1000)}{(4 \cdot 50) \cdot 300^2} = 0.713 \text{ MPa}$$

Results of MATLAB analysis

A truss analysis for a two-truss five-module bridge supporting H10-44 loading was performed using MATLAB. A simplified geometry for the truss, with 45 diagonals and concurrent members is used for the analysis. Numerical results for maximum axial force seen in each member for each loading type are shown in Table B.6 and graphical representations of the trusses with forces labelled are given in Figure B.11.

Table B.6 - Truss analysis results for 2-truss, 5-module Kenyan Truss bridge (TC/BC=Top/Bottom Chord; LD/RD=Left/Right Diagonal; V=Vertical)

	Pedestrian	Lane Loading Bending Moment	Lane Loading Shear	Design Truck
TC1-1	-65.2	-48.9	-57.4	-53.1
TC2-1	-65.2	-48.9	-57.4	-53.1
LD-1	92.1	69.2	81.2	75.1
V-1	-14.5	-26.7	-35.6	-38.6
RD-1	-71.7	-55.3	-65.4	-61.9
BC-12	115.8	84.9	99.1	90.6
TC1-2	-152.0	-111.7	-130.4	-118.1
TC2-2	-152.0	-111.7	-130.4	-118.1
LD-2	51.2	44.9	54.4	53.8
V-2	-14.5	-26.7	-35.6	-38.6
RD-2	-30.7	-31.0	-38.6	-40.6
BC-23	173.8	127.3	148.6	133.3
TC1-3	-181.0	-132.6	-154.8	-136.9
TC2-3	-181.0	-132.6	-154.8	-136.9
LD-3	10.2	20.6	27.6	32.6
V-3	-14.5	-26.7	-35.6	-38.6
RD-3	10.2	20.6	27.6	27.2
BC-34	173.8	127.3	148.6	128.2
TC1-4	-152.0	-111.7	-130.4	-109.6
TC2-4	-152.0	-111.7	-130.4	-109.6
LD-4	-30.7	-31.0	-38.6	-35.2
V-4	-14.5	-26.7	-35.6	-38.6
RD-4	51.2	44.9	54.4	48.5
BC-45	115.8	84.9	99.1	81.4
TC1-5	-65.2	-48.9	-57.4	-47.9
TC2-5	-65.2	-48.9	-57.4	-47.9
LD-5	-71.7	-55.3	-65.4	-55.4
V-5	-14.5	-26.7	-35.6	-38.6
RD-5	92.1	69.2	81.2	67.8
RX-H	0.0	0.0	0.0	0.0
RX-VL	72.4	53.0	61.9	57.2
RX-VR	72.4	53.0	61.9	50.9

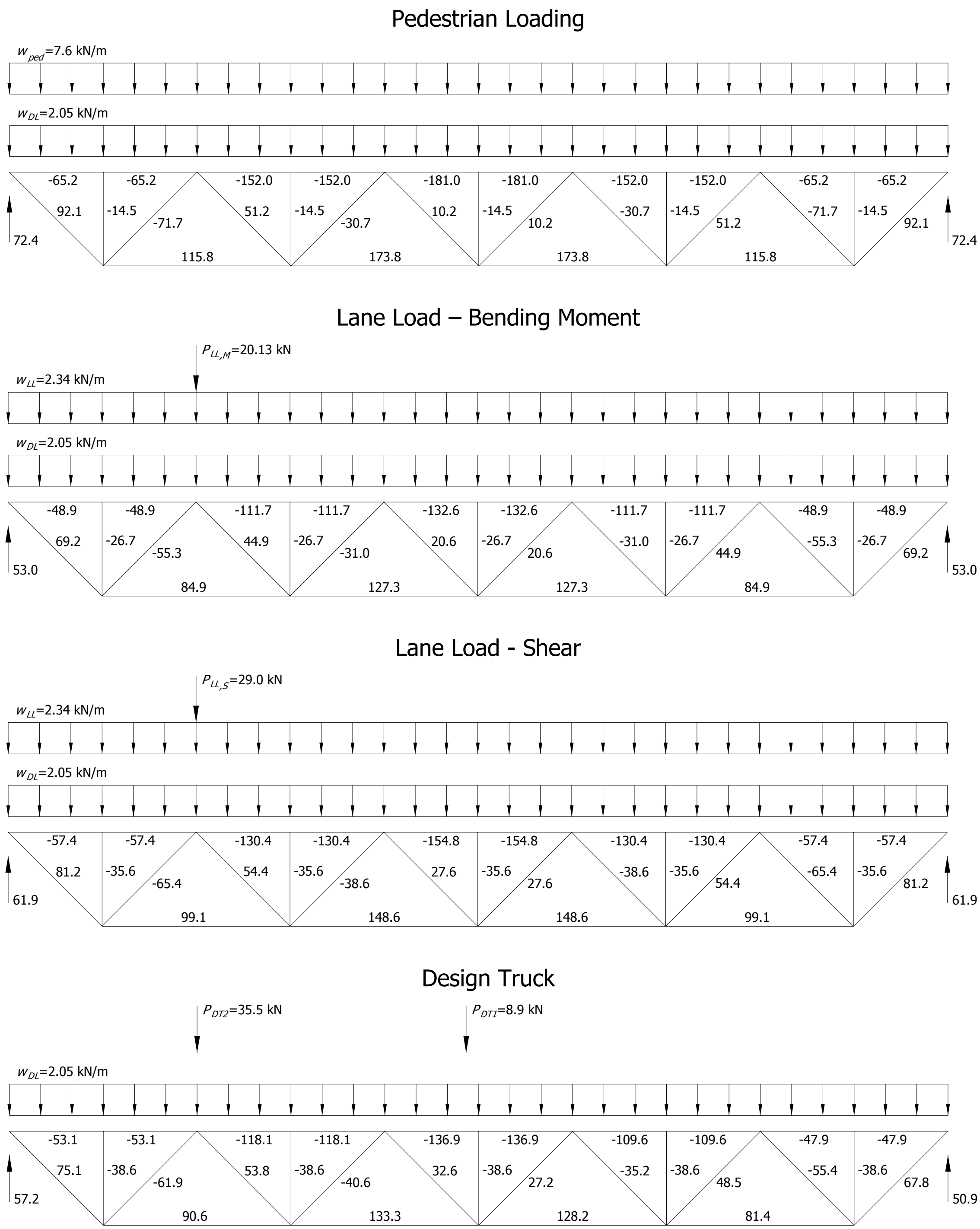


Figure B.11 - Truss analysis results for 2-truss, 5-module Kenyan Truss bridge

From the results of the analysis, the maximum force seen in each truss member type (top chord, bottom chord, diagonal, vertical) can be extracted, as these will dominate the design. Maximum forces seen for each loading type by a 2-truss 5-module bridge are given in Table B.7.

Table B.7 - Maximum forces for 2-truss, 5-module Kenyan Truss bridge

	Maximum Compression in Upper Chord (kN)	Maximum Tension in Lower Chord (kN)	Maximum Compression in Diagonals (kN)	Maximum Tension in Diagonals (kN)	Maximum Compression in Vertical (kN)
Pedestrian	181.0	173.8	71.7	92.1	14.5
Lane Load: Moment	132.6	127.3	n/a	n/a	26.7
Lane Load: Shear	n/a	n/a	65.4	81.2	35.6
Design Truck	136.9	133.3	61.9	75.1	38.6

Axial forces are converted into stresses using the cross sectional area of each member. Relevant cross-sectional areas are:

$$A_{TC} = 4 \cdot 300 \cdot 50 = 60000 \text{ mm}^2$$

$$A_{BC} = 2 \cdot 100 \cdot 6 = 1200 \text{ mm}^2$$

$$A_D = 2 \cdot 200 \cdot 50 = 20000 \text{ mm}^2$$

$$A_V = 2 \cdot 150 \cdot 50 = 15000 \text{ mm}^2$$

Final maximum stresses for pedestrian loading and the greater of lane loading or design vehicle are given in Table B.8.

Table B.8 - Design stress values for 2-truss, 5-module Kenyan Truss bridge

	Pedestrian	Maximum of Lane Loading and Design Truck
Maximum Axial Compressive Stress in Top Chord $f_{c,uc}$ (MPa)	3.02	2.28
Maximum Tensile Stress in Lower Bottom $f_{t,bc}$ (MPa)	144.80	111.09
Maximum Compressive Stress in Diagonals $f_{c,d}$ (MPa)	3.58	3.27
Maximum Tensile Stress in Diagonals $f_{t,d}$ (MPa)	4.61	4.06
Maximum Compressive Stress in Vertical $f_{c,v}$ (MPa)	0.97	2.57
Maximum Bending Stress in Upper Chord $f_{b,uc}$ (MPa)	0.71	3.61

Stresses can be primarily compared with adjusted working stress values, based on those provided by Parry (1981). Working stresses for the design are given in Table B.9. It can be noted that the values for timber correspond roughly with the values defined for Group C in Table B.1. These values are intended for structural softwood over a density of 650 kg/m3 at 18% moisture content, corresponding with the unit weight used for calculating dead load above.

Table B.9 - Working stresses for Kenyan Low-Cost Modular Timber Bridge (Parry 1981)

Design Values: MPa (ksi)	Timber
Bending, F_b	5.2 (0.754)
Tension, F_t	3.6 (0.522)
Compression parallel to the grain, F_c	5.0 (0.725)
Compression perpendicular to the grain, $F_{c\perp}$	1.16 (0.168)
Shear parallel to the grain, F_v	0.66 (0.096)
	Steel
Tension, σ_y	147 (21.3)

In addition, the combination of bending and axial compression needs to be assessed for the top chord. For uniaxial edge-wise bending, NDS (AF&PA 2005) requires:

$$\left[\frac{f_c}{F'_c}\right]^2 + \frac{f_b}{F'_b \cdot \left[1-\left(f_c / F_{cE}\right)\right]} \leq 1.0$$

where:

$$f_c < F_{cE} = \frac{0.822 \cdot E'_{\min}}{\left(l_e / d\right)^2}$$

and

l_e = effective column length = 1500 mm
d = beam depth = 300 mm
E'_{min} = adjusted minimum modulus of elasticity ~= 500 ksi = 3450 MPa

For the example case:

$$f_c = 2.24 < F_{cE} = \frac{0.822 \cdot 3450}{\left(1500/300\right)^2} = 113.4 \text{ MPa for vehicle loading}$$

and

$$f_c = 2.98 < F_{cE} = \frac{0.822 \cdot 3450}{\left(1500/300\right)^2} = 113.4 \text{ MPa for pedestrian loading}$$

These yield combination values of:

$$\left[\frac{2.28}{1.15 \cdot 5.0}\right]^2 + \frac{3.61}{1.15 \cdot 5.2 \cdot \left[1-\left(2.24/113.4\right)\right]} = 0.157 + 0.616 = 0.773 \leq 1.0$$

and

$$\left[\frac{3.02}{1.15 \cdot 5.0}\right]^2 + \frac{0.71}{1.15 \cdot 5.2 \cdot \left[1-\left(2.98/113.4\right)\right]} = 0.276 + 0.122 = 0.398 \leq 1.0$$

for vehicle loading and pedestrian loading, respectively. Both meet the design criteria, indicating that the combination of bending and axial compression is not a concern.

Final analysis results for a variety of combination bridges using the Kenyan truss system are shown in Table B.10, Table B.11, and Table B.12. Stresses that exceed allowable values are shaded. It can be seen from the results that shear forces, and particularly tension in the final diagonal, can dominate at short spans, while bottom chord tension dominates at longer spans. By comparing with Table B.5, it can also be concluded that maximum pedestrian loading was not considered in the original truss design and use recommendations. New values fore the recommended number of trusses for H10-44 and pedestrian loading are given in Table B.14.

Table B.10 - Analysis results for 2-truss Kenyan Truss bridges with lengths of 4, 5, or 6 modules

		4x2		5x2		6x2				
		Ped	Lane	Ped	Lane	Ped	Lane			
$P_{c,uc}$	kN	108.60	96.79	180.99	136.92	253.39	173.93			
$P_{t,bc}$	kN	115.84	100.68	173.75	133.31	260.63	178.85			
$P_{c,d}$	kN	51.19	54.03	71.67	65.39	92.15	76.06			
$P_{t,d}$	kN	71.67	70.59	92.15	81.23	112.62	91.45			
$P_{c,v}$	kN	14.48	38.58	14.48	38.58	14.48	38.58	Allowable Values		
$M_{max,uc}$	kN-m	2.14	10.82	2.14	10.82	2.14	10.82	C_d	1.15	1.25
$f_{c,uc}$	MPa	1.81	1.61	3.02	2.28	4.22	2.90	F'_c	5.75	6.25
$f_{t,bc}$	MPa	96.53	83.90	144.80	111.09	217.19	149.04	$F'_{t,st}$	147	147
$f_{c,d}$	MPa	2.56	2.70	3.58	3.27	4.61	3.80	F'_c	5.75	6.25
$f_{t,d}$	MPa	3.58	3.53	4.61	4.06	5.63	4.57	F'_t	4.14	4.5
$f_{c,v}$	MPa	0.97	2.57	0.97	2.57	0.97	2.57	F'_c	5.75	6.25
$f_{b,uc}$	MPa	0.71	3.61	0.71	3.61	0.71	3.61	F'_b	5.98	6.5
comb		0.22	0.69	0.40	0.77	0.66	0.87		1.0	

Table B.11 - Analysis results for 4-truss Kenyan Truss bridges with lengths of 5, 6, 7, or 8 modules

		5x4		6x4		7x4		8x4				
		Ped	Lane	Ped	Lane	Ped	Lane	Ped	Lane			
$P_{c,uc}$	kN	98.08	76.05	137.31	97.58	192.24	130.71	247.16	162.41			
$P_{t,bc}$	kN	94.16	73.94	141.24	100.35	188.32	128.05	251.09	164.98			
$P_{c,d}$	kN	38.84	35.70	49.93	41.89	61.03	47.89	72.13	53.77			
$P_{t,d}$	kN	49.93	44.48	61.03	50.44	72.13	56.28	83.22	62.04			
$P_{c,v}$	kN	7.85	19.90	7.85	19.90	7.85	19.90	7.85	19.90	Allowable Values		
$M_{max,uc}$	kN-m	1.07	5.41	1.07	5.41	1.07	5.41	1.07	5.41	C_d	1.15	1.25
$f_{c,uc}$	MPa	1.63	1.27	2.29	1.63	3.20	2.18	4.12	2.71	F'_c	5.75	6.25
$f_{t,bc}$	MPa	78.47	61.61	117.70	83.62	156.93	106.71	209.24	137.48	$F'_{t,st}$	147	147
$f_{c,d}$	MPa	1.94	1.78	2.50	2.09	3.05	2.39	3.61	2.69	F'_c	5.75	6.25
$f_{t,d}$	MPa	2.50	2.22	3.05	2.52	3.61	2.81	4.16	3.10	F'_t	4.14	4.5
$f_{c,v}$	MPa	0.52	1.33	0.52	1.33	0.52	1.33	0.52	1.33	F'_c	5.75	6.25
$f_{b,uc}$	MPa	0.36	1.80	0.36	1.80	0.36	1.80	0.36	1.80	F'_b	5.98	6.5
comb		0.14	0.35	0.22	0.39	0.37	0.45	0.58	0.53		1.0	1.0

Table B.12 - Analysis results for 6-truss Kenyan Truss bridges with lengths of 7, 8, or 9 modules

		7x6		8x6		9x6				
		Ped	Lane	Ped	Lane	Ped	Lane			
$P_{c,uc}$	kN	138.19	97.18	177.68	121.17	228.44	152.01			
$P_{t,bc}$	kN	135.37	95.19	180.50	123.09	225.62	150.13			
$P_{c,d}$	kN	43.87	35.11	51.85	39.61	59.83	44.06			
$P_{t,d}$	kN	51.85	41.29	59.83	45.70	67.80	50.09			
$P_{c,v}$	kN	5.64	13.67	5.64	13.67	5.64	13.67	Allowable Values		
$M_{max,uc}$	kN-m	0.71	3.61	0.71	3.61	0.71	3.61	C_d	1.15	1.25
$f_{c,uc}$	MPa	2.30	1.62	2.96	2.02	3.81	2.53	F'_c	5.75	6.25
$f_{t,bc}$	MPa	112.81	79.33	150.41	102.57	188.02	125.11	$F'_{t,st}$	147	147
$f_{c,d}$	MPa	2.19	1.76	2.59	1.98	2.99	2.20	F'_c	5.75	6.25
$f_{t,d}$	MPa	2.59	2.06	2.99	2.29	3.39	2.50	F'_t	4.14	4.5
$f_{c,v}$	MPa	0.38	0.91	0.38	0.91	0.38	0.91	F'_c	5.75	6.25
$f_{b,uc}$	MPa	0.24	1.20	0.24	1.20	0.24	1.20	F'_b	5.98	6.5
comb		0.20	0.28	0.31	0.33	0.48	0.40		1.0	1.0

Table B.13- Analysis results for 8-truss Kenyan Truss bridges with lengths of 8, 9, or 10 modules

		8x8		9x8		10x8				
		Ped	Lane	Ped	Lane	Ped	Lane			
$P_{c,uc}$	kN	142.88	100.51	183.71	126.39	224.53	151.71			
$P_{t,bc}$	kN	145.15	102.10	181.44	124.83	226.80	153.24			
$P_{c,d}$	kN	41.70	32.52	48.11	36.28	54.53	40.02			
$P_{t,d}$	kN	48.11	37.52	54.53	41.24	60.94	44.94			
$P_{c,v}$	kN	4.54	10.56	4.54	10.56	4.54	10.56	Allowable Values		
$M_{max,uc}$	kN-m	0.53	2.70	0.53	2.70	0.53	2.70	C_d	1.15	1.25
$f_{c,uc}$	MPa	2.38	1.68	3.06	2.11	3.74	2.53	F'_c	5.75	6.25
$f_{t,bc}$	MPa	120.96	85.08	151.20	104.02	189.00	127.70	$F'_{t,st}$	147	147
$f_{c,d}$	MPa	2.08	1.63	2.41	1.81	2.73	2.00	F'_c	5.75	6.25
$f_{t,d}$	MPa	2.41	1.88	2.73	2.06	3.05	2.25	F'_t	4.14	4.5
$f_{c,v}$	MPa	0.30	0.70	0.30	0.70	0.30	0.70	F'_c	5.75	6.25
$f_{b,uc}$	MPa	0.18	0.90	0.18	0.90	0.18	0.90	F'_b	5.98	6.5
comb		0.20	0.24	0.31	0.29	0.45	0.35		1.0	1.0

Table B.14 - Recommended number of trusses for Kenyan Low-Cost Modular Timber Bridge for pedestrian or H10-44 vehicle loading

Number of Modules	Span (m)	Number of Trusses
4	12	2
5	15	4
6	18	4
7	21	6
8	24	8

B.4 - References

AF&PA (2005). National design specification for wood construction. American Forest & Paper Association, Washington D.C.

Allotey, I. A. (1988). "Study into the behaviour of mechanically built-up timber girder for bridge construction." Materials and Structures **21**: 256-267.

Parry, J. D. (1981). The Kenyan Low Cost Modular Timber Bridge. Overseas Unit, Transport and Road Research Laboratory, Crowthorne, Berkshire, UK.

Transport Research Laboratory (2000). Overseas Road Note 9: A design manual for small bridges. Department for International Development, London, UK.

Appendix C - Connection Strength Results

C.1 - Material Properties

Bending Yield Strength

Mackay

Species	Yield Stress (psi)	5% Exclusion (psi)
Red Oak	15110	7871
White Oak	13493	7649

Schmidt and Daniels

Species	Yield Stress (psi)	5% Exclusion (psi)
White Oak	13440	7990

McFarland-Johnson (results suggested to be artificially low)

Species	Yield Stress (psi)	5% Exclusion (psi)
Red Oak	9800	
White Oak	10775	
Old White Ash	9300	

Bearing Strength - Combined Joints

Mackay

Base Material	Peg Material	Yield Stress (psi)	5% Exclusion (psi)
Douglas Fir (LT)	Red Oak	2070	1497
Douglas Fir (RT)	Red Oak	1728	1300
Eastern White Pine (LT)	Red Oak	2277	1954
Eastern White Pine (RT)	Red Oak	1469	1213

Church and Tew

Base Material	Peg Material	Yield Stress (psi)	5% Exclusion (psi)
Douglas Fir (LT)	White Oak	3027	
Douglas Fir (RT)	White Oak	1954	
Red Oak (LT)	White Oak	3088	
Red Oak (RT)	White Oak	2972	

Schmidt and Daniels

Base Material	Peg Material	Yield Stress (psi)	5% Exclusion (psi)
Red Oak (RT)	White Oak	3037	2029

Bearing Strength - Metal Dowel Bearing Strength

Soltis and Wilkinson (and used in the NDS)

$F_{e||} = 11200 \cdot G_d$ average bearing strength parallel to grain

$F_{e\perp} = 6100 \cdot G_d^{1.45} \cdot D^{-0.5}$ average bearing strength perpendicular to grain

Church and Tew (3/4" dowel diameter)

Base Material	**Yield Stress (psi)**	**5% Exclusion (psi)**
Douglas Fir (LT)	5334	
Douglas Fir (RT)	2015	
Red Oak (LT)	7726	
Red Oak (RT)	4625	

Schmidt and Daniels (1" dowel diameter)

Base Material	**Yield Stress (psi)**	**5% Exclusion (psi)**
Douglas Fir (LT)	2130	1480
Douglas Fir (RT)	6560	4580
Southern Yellow Pine (RT)	1870	1100
Red Oak (LT)	4910	4040
Red Oak (RT)	11400	7840

Bearing Strength - Peg Bearing Strength

Schmidt and Daniels (1" dowel diameter)

Peg Material	**Yield Stress (psi)**	**5% Exclusion (psi)**
White Oak	2690	1600

$F_{ed} = 5650 \cdot G_{12}^{2.04} \cong 5300 \cdot G_d^{2.04}$ (Converted using Wilkinson 1991)

Shear Strength of Dowel

Mackay (average of 3/4", 1", and 1 1/4" diameter dowel)

Peg Material	**Span**	**Yield Stress (psi)**	**5% Exclusion (psi)**
Red Oak	1/4D	1966	1489
Red Oak	1/2D	1736	1221
Red Oak	1D	1431	958
White Oak	1/4D	2250	1789
White Oak	1/2D	1965	1612
White Oak	1D	1615	1204

Schmidt and Daniels (1" dowel diameter)

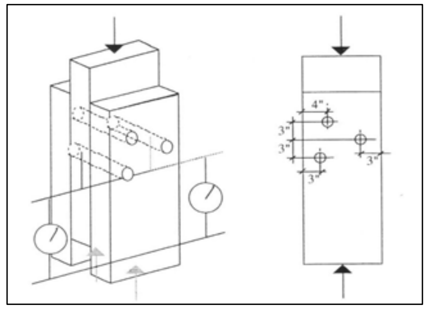
Peg Material	**Span**	**Yield Stress (psi)**	**5% Exclusion (psi)**
White Oak	1/8D	2360	1680
White Oak	1/4D	2130	1690
White Oak	1/2D	1880	1490
White Oak	1D	1560	1330

C.2 - Modified Yield Model Results

Joint Strength Prediction - McFarland-Johnson 1 Peg Parallel

Members
3" x 12" Eastern White Spruce MC=12-14%

Pegs
1-3/4" diameter White Oak MC=8-10%



Geometric Properties

Peg Diameter	D	1.75	in
Number of Pegs	n	1	
Main Member Thickness	t_m	3	in
Side Member Thickness	t_s	3	in

Material Properties

Peg Bending Strength	F_{yb}	13440	psi
Peg Shear Strength (a=1D)	F_{ev}	1560	psi
Peg Shear Strength (a=1/2D)	F_{ev}	1880	psi
Peg Shear Strength (a=1/4D)	F_{ev}	2130	psi
Peg Shear Strength (a=1/8D)	F_{ev}	2360	psi
Peg Bearing Strength	F_{ed}	2690	psi
Main Member Dowel Bearing Strength	F_{em}	4600	psi
Side Member Dowel Bearing Strength	F_{es}	4600	psi

Mode $\mathbf{I_m}$ $\quad Z = D \cdot t_m \cdot F_{em}$

Mode $\mathbf{I_s}$ $\quad Z = 2 \cdot D \cdot t_s \cdot F_{es}$

Mode $\mathbf{I_d}$ $\quad$ lesser of $Z = D \cdot t_m \cdot F_{ed}$ or $Z = 2 \cdot D \cdot t_s \cdot F_{ed}$

Mode $\mathbf{III_s}$ $\quad Z = \frac{2 \cdot D \cdot t_s \cdot F_{em} \cdot F_{es}}{2 \cdot F_{es} + F_{em}} \cdot \left(\sqrt{Q} - 1\right)$

where $Q = \frac{2 \cdot (F_{es} + F_{em})}{F_{em}} + \frac{2 \cdot F_{yb} \cdot (F_{em} + 2 \cdot F_{es}) \cdot D^2}{3 \cdot F_{em} \cdot F_{es} \cdot t_s^2}$

Mode $\mathbf{IV}$ $\quad Z = 2 \cdot D^2 \sqrt{\frac{2 \cdot F_{em} \cdot F_{es} \cdot F_{yb}}{3 \cdot (F_{es} + F_{em})}}$

Mode $\mathbf{V_d}$ $\quad Z = 2 \cdot \frac{\pi \cdot D^2}{4} \cdot F_{ev}$

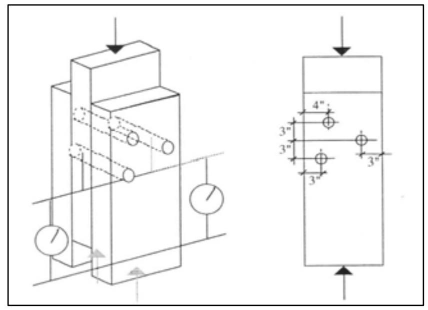
Predicted Failure Modes

Mode $\mathbf{I_m}$	24150	lbs
Mode $\mathbf{I_s}$	48300	lbs
Mode $\mathbf{I_d}$	14123	lbs
Mode $\mathbf{III_s}$	23299	lbs
Mode $\mathbf{IV}$	27805	lbs
Mode $\mathbf{V_d}$ (a=1D)	7504	lbs
Mode $\mathbf{V_d}$ (a=1/2D)	9044	lbs
Mode $\mathbf{V_d}$ (a=1/4D)	10247	lbs
Mode $\mathbf{V_d}$ (a=1/8D)	11353	lbs

Joint Strength Prediction - McFarland-Johnson 3 Peg Parallel

Members
3" x 12" Eastern White Spruce MC=12-14%

Pegs
1-3/4" diameter White Oak MC=8-10%



Mode	Equation
Mode $\mathbf{I_m}$	$Z = D \cdot t_m \cdot F_{em}$
Mode $\mathbf{I_s}$	$Z = 2 \cdot D \cdot t_s \cdot F_{es}$
Mode $\mathbf{I_d}$	lesser of $Z = D \cdot t_m \cdot F_{ed}$ or $Z = 2 \cdot D \cdot t_s \cdot F_{ed}$
Mode $\mathbf{III_s}$	$Z = \frac{2 \cdot D \cdot t_s \cdot F_{em} \cdot F_{es}}{2 \cdot F_{es} + F_{em}} \cdot \left(\sqrt{Q} - 1\right)$
	where $Q = \frac{2 \cdot (F_{es} + F_{em})}{F_{em}} + \frac{2 \cdot F_{yb} \cdot (F_{em} + 2 \cdot F_{es}) \cdot D^2}{3 \cdot F_{em} \cdot F_{es} \cdot t_s^2}$
Mode $\mathbf{IV}$	$Z = 2 \cdot D^2 \sqrt{\frac{2 \cdot F_{em} \cdot F_{es} \cdot F_{yb}}{3 \cdot (F_{es} + F_{em})}}$
Mode $\mathbf{V_d}$	$Z = 2 \cdot \frac{\pi \cdot D^2}{4} \cdot F_{ev}$

Geometric Properties

Peg Diameter	D	1.75	in
Number of Pegs	n	3	
Main Member Thickness	t_m	3	in
Side Member Thickness	t_s	3	in

Material Properties

Peg Bending Strength	F_{yb}	13440	psi
Peg Shear Strength (a=1D)	F_{ev}	1560	psi
Peg Shear Strength (a=1/2D)	F_{ev}	1880	psi
Peg Shear Strength (a=1/4D)	F_{ev}	2130	psi
Peg Shear Strength (a=1/8D)	F_{ev}	2360	psi
Peg Bearing Strength	F_{ed}	2690	psi
Main Member Dowel Bearing Strength	F_{em}	4600	psi
Side Member Dowel Bearing Strength	F_{es}	4600	psi

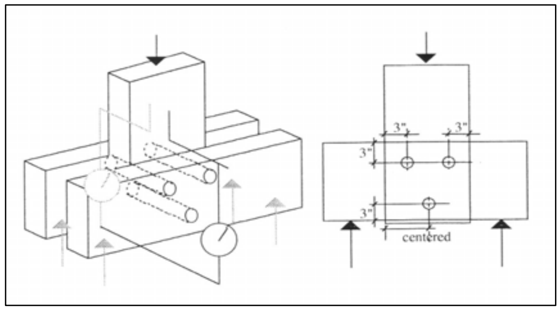
Predicted Failure Modes

Mode $\mathbf{I_m}$	72450	lbs
Mode $\mathbf{I_s}$	144900	lbs
Mode $\mathbf{I_d}$	42368	lbs
Mode $\mathbf{III_s}$	69896	lbs
Mode $\mathbf{IV}$	83415	lbs
Mode $\mathbf{V_d}$ (a=1D)	22513	lbs
Mode $\mathbf{V_d}$ (a=1/2D)	27132	lbs
Mode $\mathbf{V_d}$ (a=1/4D)	30740	lbs
Mode $\mathbf{V_d}$ (a=1/8D)	34059	lbs

Joint Strength Prediction - McFarland-Johnson 1 Peg Perpendicular

Members
3" x 12" Eastern White Spruce MC=12-14%

Pegs
1-3/4" diameter White Oak MC=8-10%



Mode $\mathbf{I_m}$ $\quad Z = D \cdot t_m \cdot F_{em}$

Mode $\mathbf{I_s}$ $\quad Z = 2 \cdot D \cdot t_s \cdot F_{es}$

Mode $\mathbf{I_d}$ $\quad$ lesser of $Z = D \cdot t_m \cdot F_{ed}$ or $Z = 2 \cdot D \cdot t_s \cdot F_{ed}$

Mode $\mathbf{III_s}$ $\quad Z = \frac{2 \cdot D \cdot t_s \cdot F_{em} \cdot F_{es}}{2 \cdot F_{es} + F_{em}} \cdot \left(\sqrt{Q} - 1\right)$

where $Q = \frac{2 \cdot (F_{es} + F_{em})}{F_{em}} + \frac{2 \cdot F_{yb} \cdot (F_{em} + 2 \cdot F_{es}) \cdot D^2}{3 \cdot F_{em} \cdot F_{es} \cdot t_s^2}$

Mode $\mathbf{IV}$ $\quad Z = 2 \cdot D^2 \sqrt{\frac{2 \cdot F_{em} \cdot F_{es} \cdot F_{yb}}{3 \cdot (F_{es} + F_{em})}}$

Mode $\mathbf{V_d}$ $\quad Z = 2 \cdot \frac{\pi \cdot D^2}{4} \cdot F_{ev}$

Geometric Properties

Peg Diameter	D	1.75	in
Number of Pegs	n	1	
Main Member Thickness	t_m	3	in
Side Member Thickness	t_s	3	in

Material Properties

Peg Bending Strength	F_{yb}	13440	psi
Peg Shear Strength (a=1D)	F_{ev}	1560	psi
Peg Shear Strength (a=1/2D)	F_{ev}	1880	psi
Peg Shear Strength (a=1/4D)	F_{ev}	2130	psi
Peg Shear Strength (a=1/8D)	F_{ev}	2360	psi
Peg Bearing Strength	F_{ed}	2690	psi
Main Member Dowel Bearing Strength	F_{em}	4600	psi
Side Member Dowel Bearing Strength	F_{es}	1266	psi

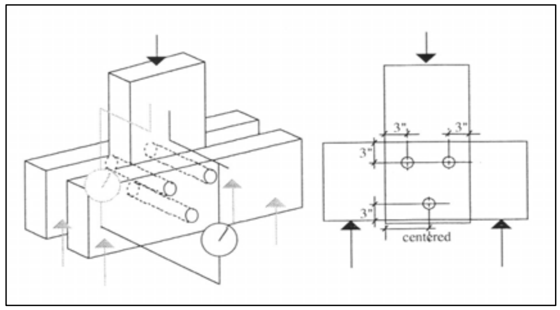
Predicted Failure Modes

Mode $\mathbf{I_m}$	24150	lbs
Mode $\mathbf{I_s}$	13290	lbs
Mode $\mathbf{I_d}$	14123	lbs
Mode $\mathbf{III_s}$	12918	lbs
Mode **IV**	18266	lbs
Mode $\mathbf{V_d}$ (a=1D)	7504	lbs
Mode $\mathbf{V_d}$ (a=1/2D)	9044	lbs
Mode $\mathbf{V_d}$ (a=1/4D)	10247	lbs
Mode $\mathbf{V_d}$ (a=1/8D)	11353	lbs

Joint Strength Prediction - McFarland-Johnson 3 Peg Perpendicular

Members
3" x 12" Eastern White Spruce MC=12-14%

Pegs
1-3/4" diameter White Oak MC=8-10%



Mode $\mathbf{I_m}$ $\quad Z = D \cdot t_m \cdot F_{em}$

Mode $\mathbf{I_s}$ $\quad Z = 2 \cdot D \cdot t_s \cdot F_{es}$

Mode $\mathbf{I_d}$ $\quad$ lesser of $Z = D \cdot t_m \cdot F_{ed}$ or $Z = 2 \cdot D \cdot t_s \cdot F_{ed}$

Mode $\mathbf{III_s}$ $\quad Z = \frac{2 \cdot D \cdot t_s \cdot F_{em} \cdot F_{es}}{2 \cdot F_{es} + F_{em}} \cdot \left(\sqrt{Q} - 1\right)$

where $Q = \frac{2 \cdot (F_{es} + F_{em})}{F_{em}} + \frac{2 \cdot F_{yb} \cdot (F_{em} + 2 \cdot F_{es}) \cdot D^2}{3 \cdot F_{em} \cdot F_{es} \cdot t_s^2}$

Mode $\mathbf{IV}$ $\quad Z = 2 \cdot D^2 \sqrt{\frac{2 \cdot F_{em} \cdot F_{es} \cdot F_{yb}}{3 \cdot (F_{es} + F_{em})}}$

Mode $\mathbf{V_d}$ $\quad Z = 2 \cdot \frac{\pi \cdot D^2}{4} \cdot F_{ev}$

Geometric Properties

Peg Diameter	D	1.75	in
Number of Pegs	n	3	
Main Member Thickness	t_m	3	in
Side Member Thickness	t_s	3	in

Material Properties

Peg Bending Strength	F_{yb}	13440	psi
Peg Shear Strength (a=1D)	F_{ev}	1560	psi
Peg Shear Strength (a=1/2D)	F_{ev}	1880	psi
Peg Shear Strength (a=1/4D)	F_{ev}	2130	psi
Peg Shear Strength (a=1/8D)	F_{ev}	2360	psi
Peg Bearing Strength	F_{ed}	2690	psi
Main Member Dowel Bearing Strength	F_{em}	4600	psi
Side Member Dowel Bearing Strength	F_{es}	1266	psi

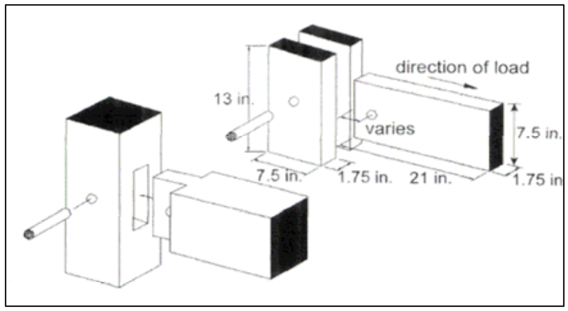
Predicted Failure Modes

Mode $\mathbf{I_m}$	72450	lbs
Mode $\mathbf{I_s}$	39871	lbs
Mode $\mathbf{I_d}$	42368	lbs
Mode $\mathbf{III_s}$	38755	lbs
Mode **IV**	54799	lbs
Mode $\mathbf{V_d}$ (a=1D)	22513	lbs
Mode $\mathbf{V_d}$ (a=1/2D)	27132	lbs
Mode $\mathbf{V_d}$ (a=1/4D)	30740	lbs
Mode $\mathbf{V_d}$ (a=1/8D)	34059	lbs

Joint Strength Prediction - Burnett 1 Peg Perpendicular - Douglas fir

Members
1.75" x 7.5" Douglas Fir Gds=0.52 Gdm=0.48

Pegs
1" diameter Red Oak Gdd=0.76



Geometric Properties

Peg Diameter	D	1	in
Number of Pegs	n	1	
Main Member Thickness	t_m	1.75	in
Side Member Thickness	t_s	1.75	in

Material Properties

Peg Bending Strength	F_{yb}	15110	psi
Peg Shear Strength (a=1D)	F_{ev}	1569	psi
Peg Shear Strength (a=1/2D)	F_{ev}	1918	psi
Peg Shear Strength (a=1/4D)	F_{ev}	2107	psi
Peg Shear Strength (a=1/8D)	F_{ev}	2366	psi
Peg Bearing Strength	F_{ed}	3028	psi
Main Member Dowel Bearing Strength	F_{em}	5376	psi
Side Member Dowel Bearing Strength	F_{es}	2363	psi

Mode $\mathbf{I_m}$ $\quad Z = D \cdot t_m \cdot F_{em}$

Mode $\mathbf{I_s}$ $\quad Z = 2 \cdot D \cdot t_s \cdot F_{es}$

Mode $\mathbf{I_d}$ $\quad$ lesser of $Z = D \cdot t_m \cdot F_{ed}$ or $Z = 2 \cdot D \cdot t_s \cdot F_{ed}$

Mode $\mathbf{III_s}$ $\quad Z = \dfrac{2 \cdot D \cdot t_s \cdot F_{em} \cdot F_{es}}{2 \cdot F_{es} + F_{em}} \cdot \left(\sqrt{Q} - 1\right)$

where $Q = \dfrac{2 \cdot (F_{es} + F_{em})}{F_{em}} + \dfrac{2 \cdot F_{yb} \cdot (F_{em} + 2 \cdot F_{es}) \cdot D^2}{3 \cdot F_{em} \cdot F_{es} \cdot t_s^2}$

Mode $\mathbf{IV}$ $\quad Z = 2 \cdot D^2 \sqrt{\dfrac{2 \cdot F_{em} \cdot F_{es} \cdot F_{yb}}{3 \cdot (F_{es} + F_{em})}}$

Mode $\mathbf{V_d}$ $\quad Z = 2 \cdot \dfrac{\pi \cdot D^2}{4} \cdot F_{ev}$

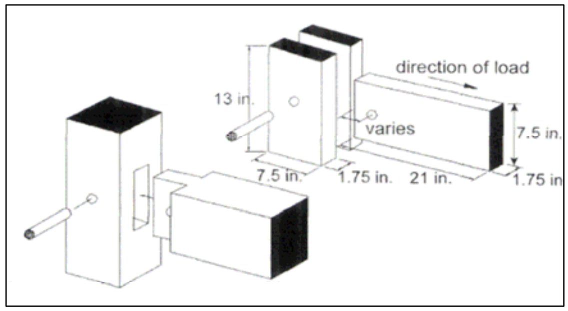
Predicted Failure Modes

Mode $\mathbf{I_m}$	9408	lbs
Mode $\mathbf{I_s}$	8272	lbs
Mode $\mathbf{I_d}$	5299	lbs
Mode $\mathbf{III_s}$	5916	lbs
Mode **IV**	8133	lbs
Mode $\mathbf{V_d}$ (a=1D)	2465	lbs
Mode $\mathbf{V_d}$ (a=1/2D)	3012	lbs
Mode $\mathbf{V_d}$ (a=1/4D)	3309	lbs
Mode $\mathbf{V_d}$ (a=1/8D)	3717	lbs

Joint Strength Prediction - Burnett 1 Peg Perpendicular - Eastern White Pine

Members
1.75" x 7.5" Eastern White Pine Gds=0.38 Gdm=0.38

Pegs
1" diameter Red Oak Gdd=0.76



Geometric Properties

Peg Diameter	D	1	in
Number of Pegs	n	1	
Main Member Thickness	t_m	1.75	in
Side Member Thickness	t_s	1.75	in

Material Properties

Peg Bending Strength	F_{yb}	15110	psi
Peg Shear Strength (a=1D)	F_{ev}	1569	psi
Peg Shear Strength (a=1/2D)	F_{ev}	1918	psi
Peg Shear Strength (a=1/4D)	F_{ev}	2107	psi
Peg Shear Strength (a=1/8D)	F_{ev}	2366	psi
Peg Bearing Strength	F_{ed}	3028	psi
Main Member Dowel Bearing Strength	F_{em}	4256	psi
Side Member Dowel Bearing Strength	F_{es}	1500	psi

Mode $\mathbf{I_m}$ $\quad Z = D \cdot t_m \cdot F_{em}$

Mode $\mathbf{I_s}$ $\quad Z = 2 \cdot D \cdot t_s \cdot F_{es}$

Mode $\mathbf{I_d}$ $\quad$ lesser of $Z = D \cdot t_m \cdot F_{ed}$ or $Z = 2 \cdot D \cdot t_s \cdot F_{ed}$

Mode $\mathbf{III_s}$ $\quad Z = \dfrac{2 \cdot D \cdot t_s \cdot F_{em} \cdot F_{es}}{2 \cdot F_{es} + F_{em}} \cdot \left(\sqrt{Q} - 1\right)$

where $Q = \dfrac{2 \cdot (F_{es} + F_{em})}{F_{em}} + \dfrac{2 \cdot F_{yb} \cdot (F_{em} + 2 \cdot F_{es}) \cdot D^2}{3 \cdot F_{em} \cdot F_{es} \cdot t_s^2}$

Mode $\mathbf{IV}$ $\quad Z = 2 \cdot D^2 \sqrt{\dfrac{2 \cdot F_{em} \cdot F_{es} \cdot F_{yb}}{3 \cdot (F_{es} + F_{em})}}$

Mode $\mathbf{V_d}$ $\quad Z = 2 \cdot \dfrac{\pi \cdot D^2}{4} \cdot F_{ev}$

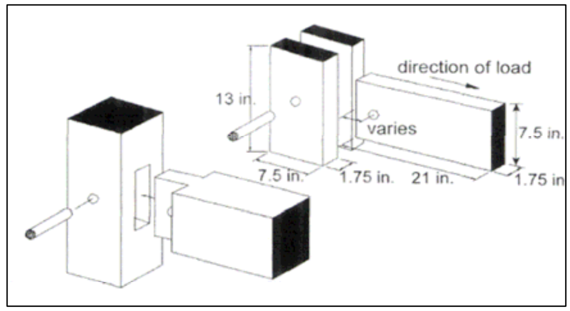
Predicted Failure Modes

Mode $\mathbf{I_m}$	7448	lbs
Mode $\mathbf{I_s}$	5249	lbs
Mode $\mathbf{I_d}$	5299	lbs
Mode $\mathbf{III_s}$	4737	lbs
Mode $\mathbf{IV}$	6685	lbs
Mode $\mathbf{V_d}$ (a=1D)	2465	lbs
Mode $\mathbf{V_d}$ (a=1/2D)	3012	lbs
Mode $\mathbf{V_d}$ (a=1/4D)	3309	lbs
Mode $\mathbf{V_d}$ (a=1/8D)	3717	lbs

Joint Strength Prediction - Burnett 1 Peg Perpendicular - Red Oak

Members
1.75" x 7.5" Red Oak Gds=0.72 Gdm=0.72

Pegs
1" diameter Red Oak Gdd=0.76



Mode $\mathbf{I_m}$ $\quad Z = D \cdot t_m \cdot F_{em}$

Mode $\mathbf{I_s}$ $\quad Z = 2 \cdot D \cdot t_s \cdot F_{es}$

Mode $\mathbf{I_d}$ $\quad$ lesser of $Z = D \cdot t_m \cdot F_{ed}$ or $Z = 2 \cdot D \cdot t_s \cdot F_{ed}$

Mode $\mathbf{III_s}$ $\quad Z = \dfrac{2 \cdot D \cdot t_s \cdot F_{em} \cdot F_{es}}{2 \cdot F_{es} + F_{em}} \cdot \left(\sqrt{Q} - 1\right)$

where $Q = \dfrac{2 \cdot (F_{es} + F_{em})}{F_{em}} + \dfrac{2 \cdot F_{yb} \cdot (F_{em} + 2 \cdot F_{es}) \cdot D^2}{3 \cdot F_{em} \cdot F_{es} \cdot t_s^2}$

Mode $\mathbf{IV}$ $\quad Z = 2 \cdot D^2 \sqrt{\dfrac{2 \cdot F_{em} \cdot F_{es} \cdot F_{yb}}{3 \cdot (F_{es} + F_{em})}}$

Mode $\mathbf{V_d}$ $\quad Z = 2 \cdot \dfrac{\pi \cdot D^2}{4} \cdot F_{ev}$

Geometric Properties

Peg Diameter	D	1	in
Number of Pegs	n	1	
Main Member Thickness	t_m	1.75	in
Side Member Thickness	t_s	1.75	in

Material Properties

Peg Bending Strength	F_{yb}	15110	psi
Peg Shear Strength (a=1D)	F_{ev}	1569	psi
Peg Shear Strength (a=1/2D)	F_{ev}	1918	psi
Peg Shear Strength (a=1/4D)	F_{ev}	2107	psi
Peg Shear Strength (a=1/8D)	F_{ev}	2366	psi
Peg Bearing Strength	F_{ed}	3028	psi
Main Member Dowel Bearing Strength	F_{em}	8064	psi
Side Member Dowel Bearing Strength	F_{es}	3788	psi

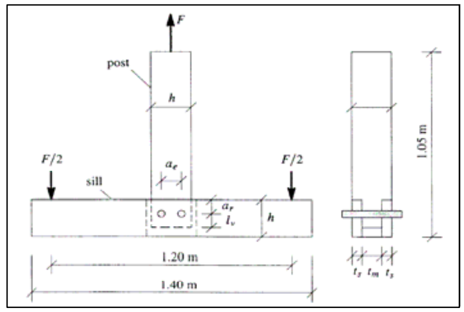
Predicted Failure Modes

Mode $\mathbf{I_m}$	14112	lbs
Mode $\mathbf{I_s}$	13260	lbs
Mode $\mathbf{I_d}$	5299	lbs
Mode $\mathbf{III_s}$	7863	lbs
Mode **IV**	10191	lbs
Mode $\mathbf{V_d}$ (a=1D)	2465	lbs
Mode $\mathbf{V_d}$ (a=1/2D)	3012	lbs
Mode $\mathbf{V_d}$ (a=1/4D)	3309	lbs
Mode $\mathbf{V_d}$ (a=1/8D)	3717	lbs

Joint Strength Prediction - Kessel and Augustin 2 Peg Perpendicular - 24mm Oak

Members
Main 140mm x 140mm Green White Oak Gdm~=0.60
Side 140mm x 45mm Green White Oak Gds~=0.60
(Specific Gravity from Wood Handbook)

Pegs
24mm diameter Cured White Oak Gdd=0.68



Geometric Properties

Peg Diameter	D	0.94	in
Number of Pegs	n	2	
Main Member Thickness	t_m	1.97	in
Side Member Thickness	t_s	1.77	in

Material Properties

Peg Bending Strength	F_{yb}	13493	psi
Peg Shear Strength (a=1D)	F_{ev}	1475	psi
Peg Shear Strength (a=1/2D)	F_{ev}	1747	psi
Peg Shear Strength (a=1/4D)	F_{ev}	1910	psi
Peg Shear Strength (a=1/8D)	F_{ev}	2103	psi
Peg Bearing Strength	F_{ed}	2413	psi
Main Member Dowel Bearing Strength	F_{em}	6720	psi
Side Member Dowel Bearing Strength	F_{es}	2992	psi

Mode $\mathbf{I_m}$ $\quad Z = D \cdot t_m \cdot F_{em}$

Mode $\mathbf{I_s}$ $\quad Z = 2 \cdot D \cdot t_s \cdot F_{es}$

Mode $\mathbf{I_d}$ $\quad$ lesser of $Z = D \cdot t_m \cdot F_{ed}$ or $Z = 2 \cdot D \cdot t_s \cdot F_{ed}$

Mode $\mathbf{III_s}$ $\quad Z = \frac{2 \cdot D \cdot t_s \cdot F_{em} \cdot F_{es}}{2 \cdot F_{es} + F_{em}} \cdot \left(\sqrt{Q} - 1\right)$

where $Q = \frac{2 \cdot (F_{es} + F_{em})}{F_{em}} + \frac{2 \cdot F_{yb} \cdot (F_{em} + 2 \cdot F_{es}) \cdot D^2}{3 \cdot F_{em} \cdot F_{es} \cdot t_s^2}$

Mode $\mathbf{IV}$ $\quad Z = 2 \cdot D^2 \sqrt{\frac{2 \cdot F_{em} \cdot F_{es} \cdot F_{yb}}{3 \cdot (F_{es} + F_{em})}}$

Mode $\mathbf{V_d}$ $\quad Z = 2 \cdot \frac{\pi \cdot D^2}{4} \cdot F_{ev}$

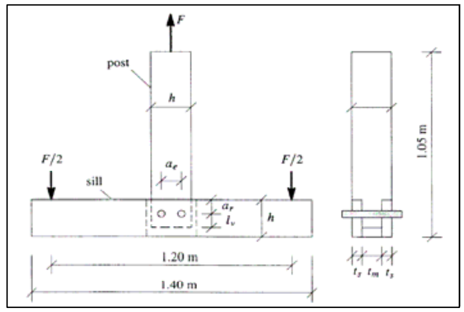
Predicted Failure Modes

Mode $\mathbf{I_m}$	24998	lbs
Mode $\mathbf{I_s}$	20034	lbs
Mode $\mathbf{I_d}$	8977	lbs
Mode $\mathbf{III_s}$	11901	lbs
Mode **IV**	15411	lbs
Mode $\mathbf{V_d}$ (a=1D)	4136	lbs
Mode $\mathbf{V_d}$ (a=1/2D)	4899	lbs
Mode $\mathbf{V_d}$ (a=1/4D)	5358	lbs
Mode $\mathbf{V_d}$ (a=1/8D)	5898	lbs

Joint Strength Prediction - Kessel and Augustin 2 Peg Perpendicular - 32mm Oak

Members
Main 200mm x 200mm Green White Oak Gdm~=0.60
Side 200mm x 60mm Green White Oak Gds~=0.60
(Specific Gravity from Wood Handbook)

Pegs
24mm diameter Cured White Oak Gdd=0.68



Mode $\mathbf{I_m}$ $\quad Z = D \cdot t_m \cdot F_{em}$

Mode $\mathbf{I_s}$ $\quad Z = 2 \cdot D \cdot t_s \cdot F_{es}$

Mode $\mathbf{I_d}$ $\quad$ lesser of $Z = D \cdot t_m \cdot F_{ed}$ or $Z = 2 \cdot D \cdot t_s \cdot F_{ed}$

Mode $\mathbf{III_s}$ $\quad Z = \frac{2 \cdot D \cdot t_s \cdot F_{em} \cdot F_{es}}{2 \cdot F_{es} + F_{em}} \cdot \left(\sqrt{Q} - 1\right)$

where $Q = \frac{2 \cdot (F_{es} + F_{em})}{F_{em}} + \frac{2 \cdot F_{yb} \cdot (F_{em} + 2 \cdot F_{es}) \cdot D^2}{3 \cdot F_{em} \cdot F_{es} \cdot t_s^2}$

Mode $\mathbf{IV}$ $\quad Z = 2 \cdot D^2 \sqrt{\frac{2 \cdot F_{em} \cdot F_{es} \cdot F_{yb}}{3 \cdot (F_{es} + F_{em})}}$

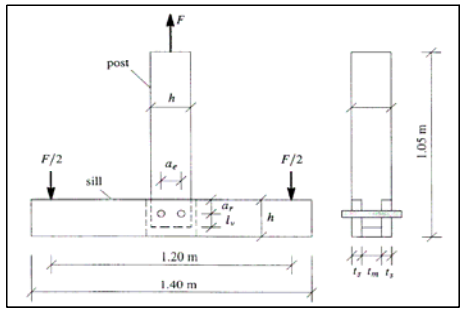
Mode $\mathbf{V_d}$ $\quad Z = 2 \cdot \frac{\pi \cdot D^2}{4} \cdot F_{ev}$

Geometric Properties			
Peg Diameter	D	1.26	in
Number of Pegs	n	2	
Main Member Thickness	t_m	3.15	in
Side Member Thickness	t_s	2.36	in

Material Properties			
Peg Bending Strength	F_{yb}	13493	psi
Peg Shear Strength (a=1D)	F_{ev}	1475	psi
Peg Shear Strength (a=1/2D)	F_{ev}	1747	psi
Peg Shear Strength (a=1/4D)	F_{ev}	1910	psi
Peg Shear Strength (a=1/8D)	F_{ev}	2103	psi
Peg Bearing Strength	F_{ed}	2413	psi
Main Member Dowel Bearing Strength	F_{em}	6720	psi
Side Member Dowel Bearing Strength	F_{es}	2591	psi

Predicted Failure Modes		
Mode $\mathbf{I_m}$	53330	lbs
Mode $\mathbf{I_s}$	30845	lbs
Mode $\mathbf{I_d}$	19151	lbs
Mode $\mathbf{III_s}$	19610	lbs
Mode **IV**	26039	lbs
Mode $\mathbf{V_d}$ (a=1D)	7353	lbs
Mode $\mathbf{V_d}$ (a=1/2D)	8710	lbs
Mode $\mathbf{V_d}$ (a=1/4D)	9525	lbs
Mode $\mathbf{V_d}$ (a=1/8D)	10486	lbs

Joint Strength Prediction - Kessel and Augustin 2 Peg Perpendicular - 40mm Oak



Members
Main 200mm x 200mm Green White Oak Gdm~=0.60
Side 200mm x 60mm Green White Oak Gds~=0.60
(Specific Gravity from Wood Handbook)

Pegs
24mm diameter Cured White Oak Gdd=0.68

Geometric Properties

Peg Diameter	D	1.57	in
Number of Pegs	n	2	
Main Member Thickness	t_m	3.15	in
Side Member Thickness	t_s	2.36	in

Material Properties

Peg Bending Strength	F_{yb}	13493	psi
Peg Shear Strength (a=1D)	F_{ev}	1475	psi
Peg Shear Strength (a=1/2D)	F_{ev}	1747	psi
Peg Shear Strength (a=1/4D)	F_{ev}	1910	psi
Peg Shear Strength (a=1/8D)	F_{ev}	2103	psi
Peg Bearing Strength	F_{ed}	2413	psi
Main Member Dowel Bearing Strength	F_{em}	6720	psi
Side Member Dowel Bearing Strength	F_{es}	2318	psi

Mode $\mathbf{I_m}$ $\quad Z = D \cdot t_m \cdot F_{em}$

Mode $\mathbf{I_s}$ $\quad Z = 2 \cdot D \cdot t_s \cdot F_{es}$

Mode $\mathbf{I_d}$ $\quad$ lesser of $Z = D \cdot t_m \cdot F_{ed}$ or $Z = 2 \cdot D \cdot t_s \cdot F_{ed}$

Mode $\mathbf{III_s}$ $\quad Z = \dfrac{2 \cdot D \cdot t_s \cdot F_{em} \cdot F_{es}}{2 \cdot F_{es} + F_{em}} \cdot \left(\sqrt{Q} - 1\right)$

where $Q = \dfrac{2 \cdot (F_{es} + F_{em})}{F_{em}} + \dfrac{2 \cdot F_{yb} \cdot (F_{em} + 2 \cdot F_{es}) \cdot D^2}{3 \cdot F_{em} \cdot F_{es} \cdot t_s^2}$

Mode $\mathbf{IV}$ $\quad Z = 2 \cdot D^2 \sqrt{\dfrac{2 \cdot F_{em} \cdot F_{es} \cdot F_{yb}}{3 \cdot (F_{es} + F_{em})}}$

Mode $\mathbf{V_d}$ $\quad Z = 2 \cdot \dfrac{\pi \cdot D^2}{4} \cdot F_{ev}$

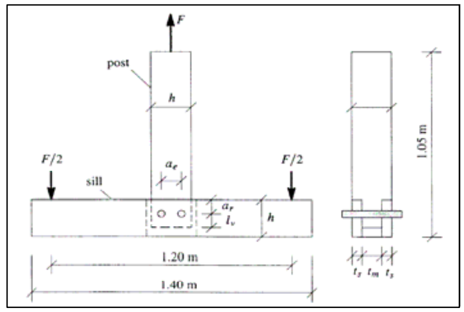
Predicted Failure Modes

Mode $\mathbf{I_m}$	66663	lbs
Mode $\mathbf{I_s}$	34486	lbs
Mode $\mathbf{I_d}$	23939	lbs
Mode $\mathbf{III_s}$	27907	lbs
Mode $\mathbf{IV}$	39057	lbs
Mode $\mathbf{V_d}$ (a=1D)	11488	lbs
Mode $\mathbf{V_d}$ (a=1/2D)	13609	lbs
Mode $\mathbf{V_d}$ (a=1/4D)	14882	lbs
Mode $\mathbf{V_d}$ (a=1/8D)	16385	lbs

Joint Strength Prediction - Kessel and Augustin 2 Peg Perpendicular - 24mm Spruce

Members
Main 140mm x 140mm Dry Spruce Gdm~=0.40
Side 140mm x 45mm Dry Spruce Gds~=0.40
(Specific Gravity from Wood Handbook)

Pegs
24mm diameter Cured White Oak Gdd=0.68



Mode $\mathbf{I_m}$ $\quad Z = D \cdot t_m \cdot F_{em}$

Mode $\mathbf{I_s}$ $\quad Z = 2 \cdot D \cdot t_s \cdot F_{es}$

Mode $\mathbf{I_d}$ $\quad$ lesser of $Z = D \cdot t_m \cdot F_{ed}$ or $Z = 2 \cdot D \cdot t_s \cdot F_{ed}$

Mode $\mathbf{III_s}$ $\quad Z = \frac{2 \cdot D \cdot t_s \cdot F_{em} \cdot F_{es}}{2 \cdot F_{es} + F_{em}} \cdot \left(\sqrt{Q} - 1\right)$

where $Q = \frac{2 \cdot (F_{es} + F_{em})}{F_{em}} + \frac{2 \cdot F_{yb} \cdot (F_{em} + 2 \cdot F_{es}) \cdot D^2}{3 \cdot F_{em} \cdot F_{es} \cdot t_s^2}$

Mode $\mathbf{IV}$ $\quad Z = 2 \cdot D^2 \sqrt{\frac{2 \cdot F_{em} \cdot F_{es} \cdot F_{yb}}{3 \cdot (F_{es} + F_{em})}}$

Mode $\mathbf{V_d}$ $\quad Z = 2 \cdot \frac{\pi \cdot D^2}{4} \cdot F_{ev}$

Geometric Properties

Peg Diameter	D	0.94	in
Number of Pegs	n	2	
Main Member Thickness	t_m	1.97	in
Side Member Thickness	t_s	1.77	in

Material Properties

Peg Bending Strength	F_{yb}	13493	psi
Peg Shear Strength (a=1D)	F_{ev}	1475	psi
Peg Shear Strength (a=1/2D)	F_{ev}	1747	psi
Peg Shear Strength (a=1/4D)	F_{ev}	1910	psi
Peg Shear Strength (a=1/8D)	F_{ev}	2103	psi
Peg Bearing Strength	F_{ed}	2413	psi
Main Member Dowel Bearing Strength	F_{em}	4480	psi
Side Member Dowel Bearing Strength	F_{es}	1662	psi

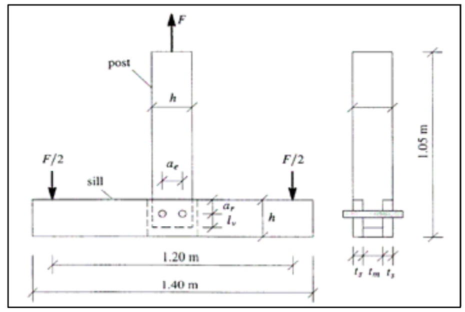
Predicted Failure Modes

Mode $\mathbf{I_m}$	16666	lbs
Mode $\mathbf{I_s}$	11129	lbs
Mode $\mathbf{I_d}$	8977	lbs
Mode $\mathbf{III_s}$	8490	lbs
Mode $\mathbf{IV}$	11793	lbs
Mode $\mathbf{V_d}$ (a=1D)	4136	lbs
Mode $\mathbf{V_d}$ (a=1/2D)	4899	lbs
Mode $\mathbf{V_d}$ (a=1/4D)	5358	lbs
Mode $\mathbf{V_d}$ (a=1/8D)	5898	lbs

Joint Strength Prediction - Kessel and Augustin 2 Peg Perpendicular - 32mm Spruce

Members
Main 200mm x 200mm Dry Spruce Gdm~=0.40
Side 200mm x 60mm Dry Spruce Gds~=0.40
(Specific Gravity from Wood Handbook)

Pegs
24mm diameter Cured White Oak Gdd=0.68



Mode $\mathbf{I_m}$ $\quad Z = D \cdot t_m \cdot F_{em}$

Mode $\mathbf{I_s}$ $\quad Z = 2 \cdot D \cdot t_s \cdot F_{es}$

Mode $\mathbf{I_d}$ $\quad$ lesser of $Z = D \cdot t_m \cdot F_{ed}$ or $Z = 2 \cdot D \cdot t_s \cdot F_{ed}$

Mode $\mathbf{III_s}$ $\quad Z = \frac{2 \cdot D \cdot t_s \cdot F_{em} \cdot F_{es}}{2 \cdot F_{es} + F_{em}} \cdot \left(\sqrt{Q} - 1\right)$

where $Q = \frac{2 \cdot (F_{es} + F_{em})}{F_{em}} + \frac{2 \cdot F_{yb} \cdot (F_{em} + 2 \cdot F_{es}) \cdot D^2}{3 \cdot F_{em} \cdot F_{es} \cdot t_s^2}$

Mode $\mathbf{IV}$ $\quad Z = 2 \cdot D^2 \sqrt{\frac{2 \cdot F_{em} \cdot F_{es} \cdot F_{yb}}{3 \cdot (F_{es} + F_{em})}}$

Mode $\mathbf{V_d}$ $\quad Z = 2 \cdot \frac{\pi \cdot D^2}{4} \cdot F_{ev}$

Geometric Properties

Peg Diameter	D	1.26	in
Number of Pegs	n	2	
Main Member Thickness	t_m	3.15	in
Side Member Thickness	t_s	2.36	in

Material Properties

Peg Bending Strength	F_{yb}	13493	psi
Peg Shear Strength (a=1D)	F_{ev}	1475	psi
Peg Shear Strength (a=1/2D)	F_{ev}	1747	psi
Peg Shear Strength (a=1/4D)	F_{ev}	1910	psi
Peg Shear Strength (a=1/8D)	F_{ev}	2103	psi
Peg Bearing Strength	F_{ed}	2413	psi
Main Member Dowel Bearing Strength	F_{em}	4480	psi
Side Member Dowel Bearing Strength	F_{es}	1439	psi

Predicted Failure Modes

Mode $\mathbf{I_m}$	35553	lbs
Mode $\mathbf{I_s}$	17134	lbs
Mode $\mathbf{I_d}$	19151	lbs
Mode $\mathbf{III_s}$	14171	lbs
Mode **IV**	19874	lbs
Mode $\mathbf{V_d}$ (a=1D)	7353	lbs
Mode $\mathbf{V_d}$ (a=1/2D)	8710	lbs
Mode $\mathbf{V_d}$ (a=1/4D)	9525	lbs
Mode $\mathbf{V_d}$ (a=1/8D)	10486	lbs

Appendix D - Yield Mode Comparison Factor Calculations

D.1 - Mode Yield Equations

Mode $\mathbf{I_m}$ $\quad ZI_m = D \cdot t \cdot 11200 \cdot G_{dm}$

Mode $\mathbf{I_d}$ $\quad ZI_d = D \cdot t \cdot 5300 \cdot G_{dd}^{2.04}$

Mode $\mathbf{III_s}$ $\quad ZIII = 7467 \cdot D \cdot t \cdot G_{dm} \cdot \left(\sqrt{Q} - 1\right)$ where $Q = 4 + \dfrac{47660 G_{dd}^{1.87} \cdot D^2}{11200 G_{dm} \cdot t^2}$

Mode $\mathbf{IV}$ $\quad ZIV = 18864 \cdot D^2 \cdot G_{dm}^{0.5} \cdot G_{dd}^{0.935}$

Mode $\mathbf{V_d}$ $\quad ZV_d = \dfrac{\pi \cdot D^2}{2} \cdot 2415 \cdot G_{dd}^{0.84}$ for $a = 1/2 \cdot D$

D.2 - Comparison Factors

Comparison Factor 1:

$\mathbf{I_m}$ vs $\mathbf{I_d}$ $\quad CF_1 = \dfrac{ZI_m}{ZI_d} = \dfrac{D \cdot t \cdot 11200 \cdot G_{dm}}{D \cdot t \cdot 5300 \cdot G_{dd}^{2.04}} = 2.113 \cdot \dfrac{G_{dm}}{G_{dd}^{2.04}}$

Comparison Factor 2:

$\mathbf{I_m}$ vs $\mathbf{V_d}$ $\quad CF_2 = \dfrac{ZI_m}{ZV_d} = \dfrac{2 \cdot D \cdot t \cdot 11200 \cdot G_{dm}}{\pi \cdot D^2 \cdot 2415 \cdot G_{dd}^{0.84}} = 2.952 \cdot \dfrac{t \cdot G_{dm}}{D \cdot G_{dd}^{0.84}}$

Comparison Factor 3:

$\mathbf{I_d}$ vs $\mathbf{V_d}$ $\quad CF_3 = \dfrac{ZI_d}{ZV_d} = \dfrac{2 \cdot D \cdot t \cdot 5300 \cdot G_{dd}^{2.04}}{\pi \cdot D^2 \cdot 2415 \cdot G_{dd}^{0.84}} = 1.397 \cdot \dfrac{t}{D} \cdot G_{dd}^{1.20}$

Comparison Factor 4:

$\mathbf{IV}$ vs $\mathbf{V_d}$ $\quad CF_4 = \dfrac{ZIV}{ZV_d} = \dfrac{2 \cdot 18864 \cdot D^2 \cdot G_{dm}^{0.5} \cdot G_{dd}^{0.935}}{\pi \cdot D^2 \cdot 2415 \cdot G_{dd}^{0.84}} = 4.972 \cdot G_{dm}^{0.5} \cdot G_{dd}^{0.095}$

Comparison Factor 5:

$\mathbf{I_m}$ vs $\mathbf{IV}$ $\quad CF_5 = \dfrac{ZI_m}{ZIV} = \dfrac{D \cdot t \cdot 11200 \cdot G_{dm}}{18864 \cdot D^2 \sqrt{G_{dm} \cdot G_{dd}^{1.87}}} = 0.5937 \cdot \dfrac{t}{D} \dfrac{G_{dm}^{0.5}}{G_{dd}^{0.935}}$

Comparison Factor 6:

$\mathbf{I_d}$ vs $\mathbf{IV}$ $\quad CF_6 = \dfrac{ZI_d}{ZIV} = \dfrac{D \cdot t \cdot 5300 \cdot G_{dd}^{2.04}}{18864 \cdot D^2 \sqrt{G_{dm} \cdot G_{dd}^{1.87}}} = 0.2810 \cdot \dfrac{t}{D} \dfrac{G_{dd}^{1.105}}{G_{dm}^{0.5}}$

Comparison Factor 7:

$\mathbf{I_m}$ vs **III**

$$CF_7 = \frac{ZI_m}{ZIII} = \frac{D \cdot t \cdot 11200 \cdot G_{dm}}{7467 \cdot D \cdot t \cdot G_{dm} \cdot \left(\sqrt{4 + \frac{47660 G_{dd}^{1.87} \cdot D^2}{11200 G_{dm} \cdot t^2}} - 1 \right)} = \frac{1.50}{\sqrt{4 + \frac{47660 G_{dd}^{1.87} \cdot D^2}{11200 G_{dm} \cdot t^2}} - 1}$$

Comparison Factor 8:

$\mathbf{I_d}$ vs **III**

$$CF_8 = \frac{ZI_d}{ZIII} = \frac{D \cdot t \cdot 5300 \cdot G_{dd}^{2.04}}{7467 \cdot D \cdot t \cdot G_{dm} \cdot \left(\sqrt{4 + \frac{47660 G_{dd}^{1.87} \cdot D^2}{11200 G_{dm} \cdot t^2}} - 1 \right)} = \frac{0.710}{\sqrt{4 + \frac{47660 G_{dd}^{1.87} \cdot D^2}{11200 G_{dm} \cdot t^2}} - 1} \cdot \frac{G_{dd}^{2.04}}{G_{dm}}$$

Comparison Factor 9:

III vs **$\mathbf{V_d}$**

$$CF_9 = \frac{ZIII}{ZV_d} = \frac{2 \cdot 7467 \cdot D \cdot t \cdot G_{dm} \cdot \left(\sqrt{4 + \frac{47660 G_{dd}^{1.87} \cdot D^2}{11200 G_{dm} \cdot t^2}} - 1 \right)}{\pi \cdot D^2 \cdot 2415 \cdot G_{dd}^{0.84}}$$

$$= 1.968 \cdot \frac{G_{dm}}{G_{dd}^{0.84}} \cdot \frac{t}{D} \cdot \left(\sqrt{4 + \frac{47660 G_{dd}^{1.87} \cdot D^2}{11200 G_{dm} \cdot t^2}} - 1 \right)$$

D.3 - Yield Mode Equalities

For each of the comparison factors derived above, the first mode will dominate if CF <1 and the second mode will dominate if CF >1. Setting the CF = 1 allows us to find the transition. Solving for G_{dd} as a function of G_{dm} and D/t

Comparison Factor 1:

$$CF_1 = 1 = 2.113 \cdot \frac{G_{dm}}{G_{dd}^{2.04}}, \; G_{dm} = 0.473 \cdot G_{dd}^{2.04}, \; G_{dd} = 1.443 \cdot G_{dm}^{0.49}$$

Comparison Factor 2:

$$CF_2 = 1 = 2.952 \cdot \frac{t \cdot G_{dm}}{D \cdot G_{dd}^{0.84}}, \; G_{dd} = 3.628 \cdot G_{dm}^{1.190} \cdot \left(\frac{D}{t} \right)^{-1.190}$$

Comparison Factor 3:

$$CF_3 = 1 = 1.397 \cdot \frac{t}{D} \cdot G_{dd}^{1.20}, \; G_{dd} = 0.757 \left(\frac{D}{t} \right)^{0.833}$$

Comparison Factor 4:

$$CF_4 = 1 = 4.972 \cdot G_{dm}^{0.5} \cdot G_{dd}^{0.095}, G_{dd} = 4.66 \cdot 10^{-8} \cdot G_{dm}^{-5.263}$$

Comparison Factor 5:

$$CF_5 = 1 = 0.5937 \cdot \frac{t}{D} \frac{G_{dm}^{0.5}}{G_{dd}^{0.935}}, \; G_{dd} = 0.573 \cdot G_{dm}^{0.533} \cdot \left(\frac{D}{t}\right)^{-0.935}$$

Comparison Factor 6:

$$CF_6 = 1 = 0.2810 \cdot \frac{t}{D} \frac{G_{dd}^{1.105}}{G_{dm}^{0.5}}, \; G_{dd} = 3.154 \cdot G_{dm}^{0.452} \cdot \left(\frac{D}{t}\right)^{0.905}$$

Comparison Factor 7:

$$CF_7 = 1 = \frac{1.50}{\sqrt{4 + \frac{47660 G_{dd}^{1.87} \cdot D^2}{11200 G_{dm} \cdot t^2}} - 1}, G_{dd} = 0.2828 \cdot G_{dm}^{0.534} \cdot \left(\frac{D}{t}\right)^{-1.070}$$

Comparison Factor 8:

$$CF_8 = 1 = \frac{0.710}{\sqrt{4 + \frac{47660 G_{dd}^{1.87} \cdot D^2}{11200 G_{dm} \cdot t^2}} - 1} \cdot \frac{G_{dd}^{2.04}}{G_{dm}}$$

$$\sqrt{4 + \frac{47660 G_{dd}^{1.87} \cdot D^2}{11200 G_{dm} \cdot t^2}} = 0.710 \cdot \frac{G_{dd}^{2.04}}{G_{dm}} + 1$$

$$0.5041 \cdot \frac{G_{dd}^{4.08}}{G_{dm}} + 1.42 \cdot G_{dd}^{2.04} - 3 \cdot G_{dm} - 4.255 \cdot \left(\frac{D}{t}\right)^2 \cdot G_{dd}^{1.87} = 0$$

Comparison Factor 9:

$$CF_9 = 1 = 1.968 \cdot \frac{G_{dm}}{G_{dd}^{0.84}} \cdot \frac{t}{D} \cdot \left(\sqrt{4 + \frac{47660 G_{dd}^{1.87} \cdot D^2}{11200 G_{dm} \cdot t^2}} - 1\right)$$

$$\sqrt{4 + \frac{47660 G_{dd}^{1.87} \cdot D^2}{11200 G_{dm} \cdot t^2}} = 0.508 \cdot \frac{G_{dd}^{0.84}}{G_{dm}} \cdot \frac{D}{t} + 1$$

$$0.258 \cdot \frac{G_{dd}^{1.12}}{G_{dm}} \cdot \left(\frac{D}{t}\right)^2 + 1.016 \cdot G_{dd}^{0.56} \cdot \frac{D}{t} - 3 G_{dm} - 4.255 \cdot \left(\frac{D}{t}\right)^2 \cdot G_{dd}^{1.87} = 0$$

Appendix E - Chord Termination Patterns

E.1 - Pattern Diagrams

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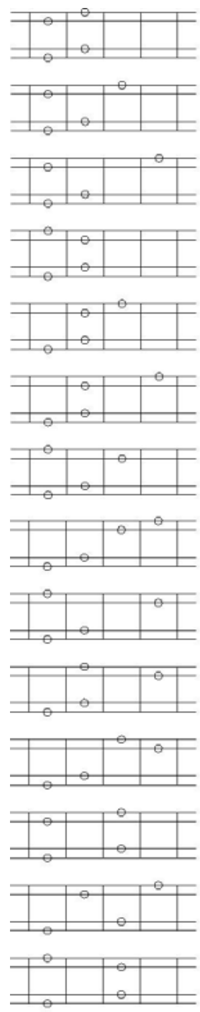

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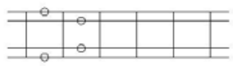
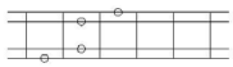
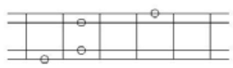
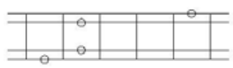
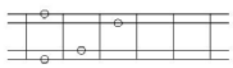
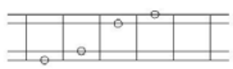


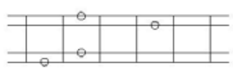
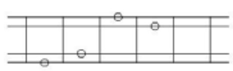
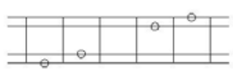
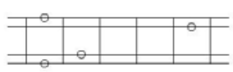
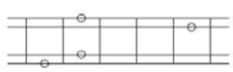
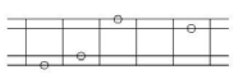
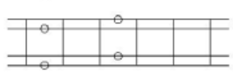
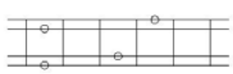
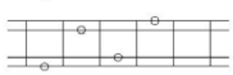
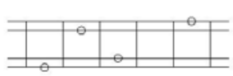
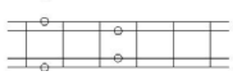
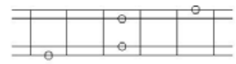

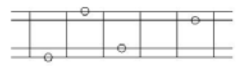

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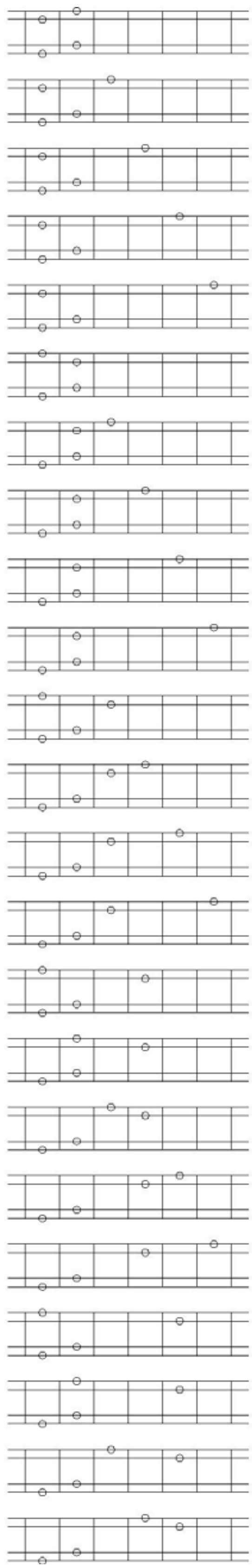

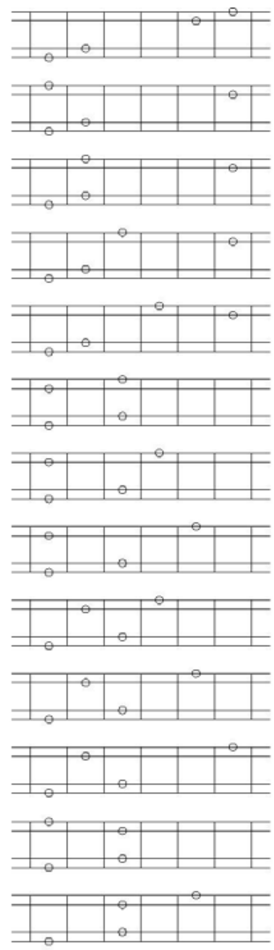

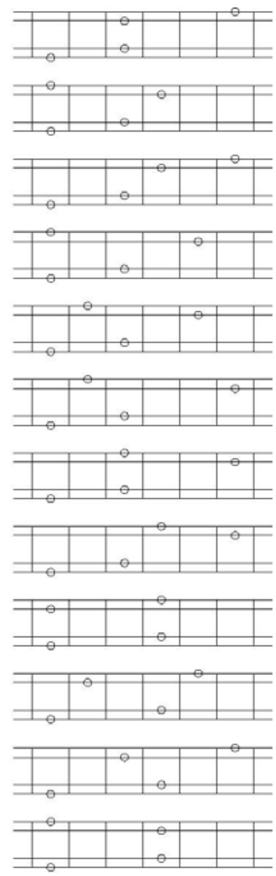

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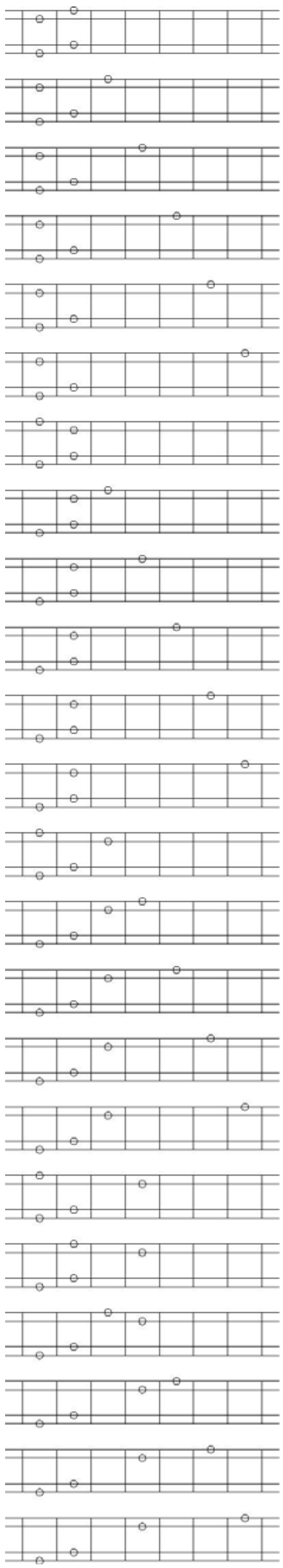

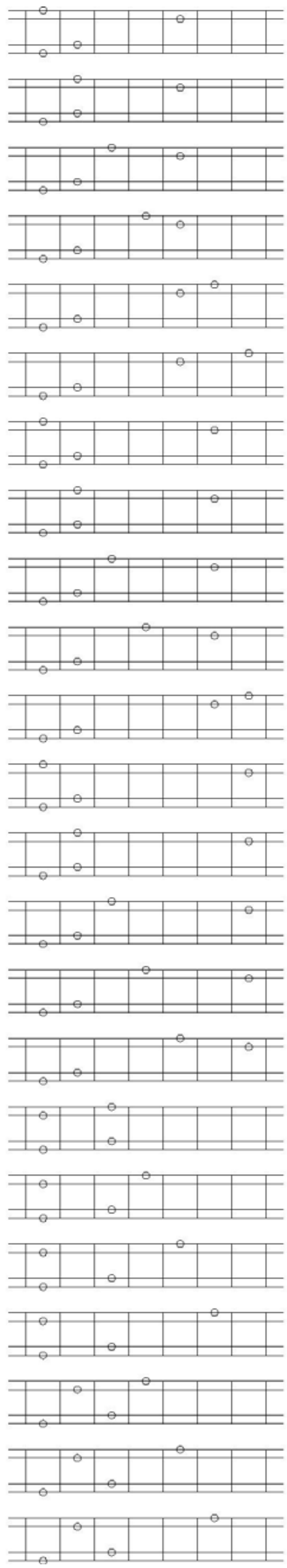

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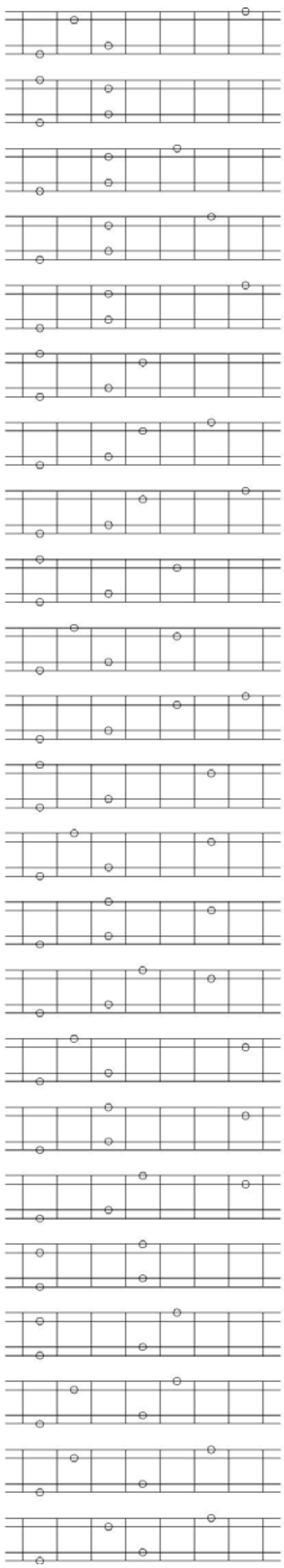

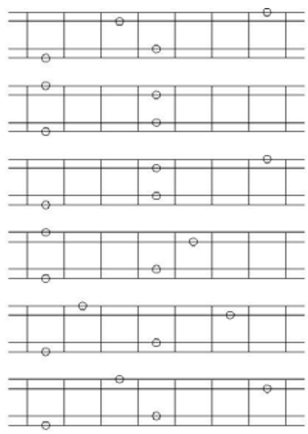

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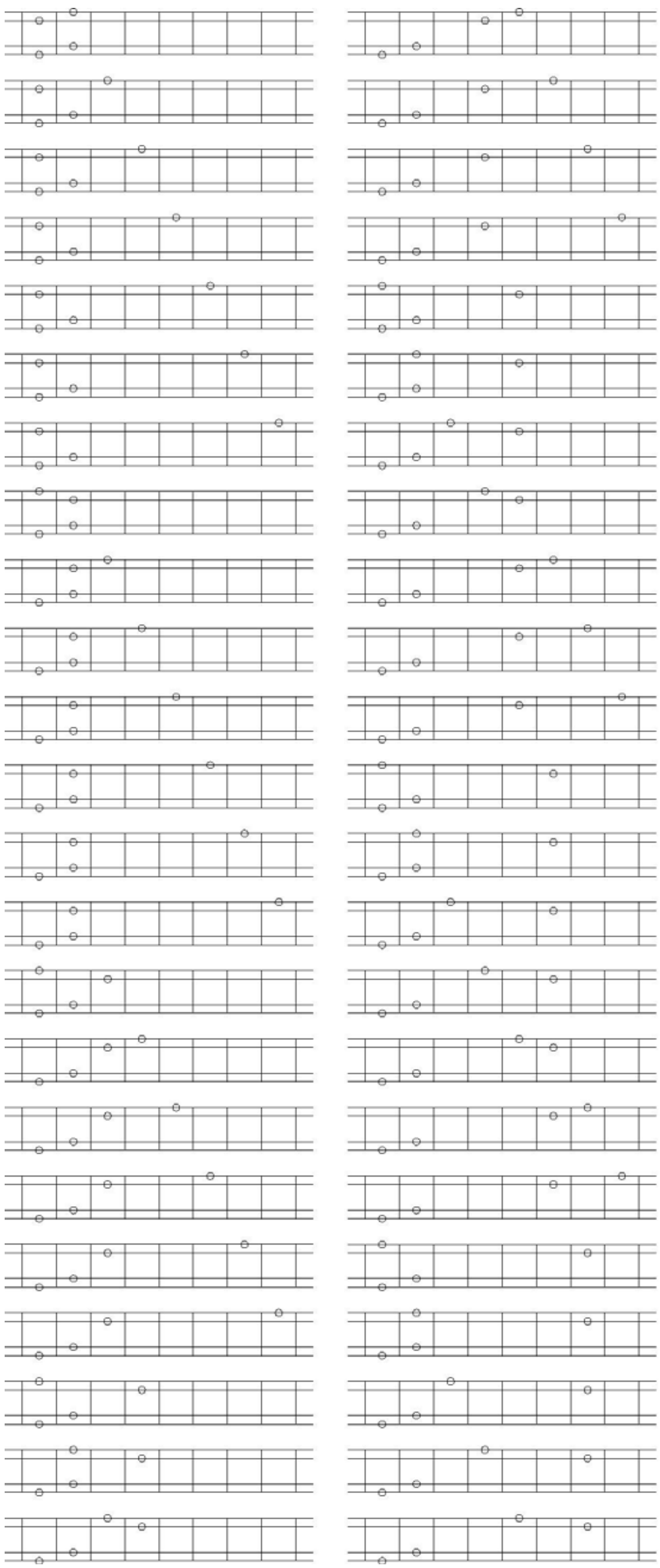

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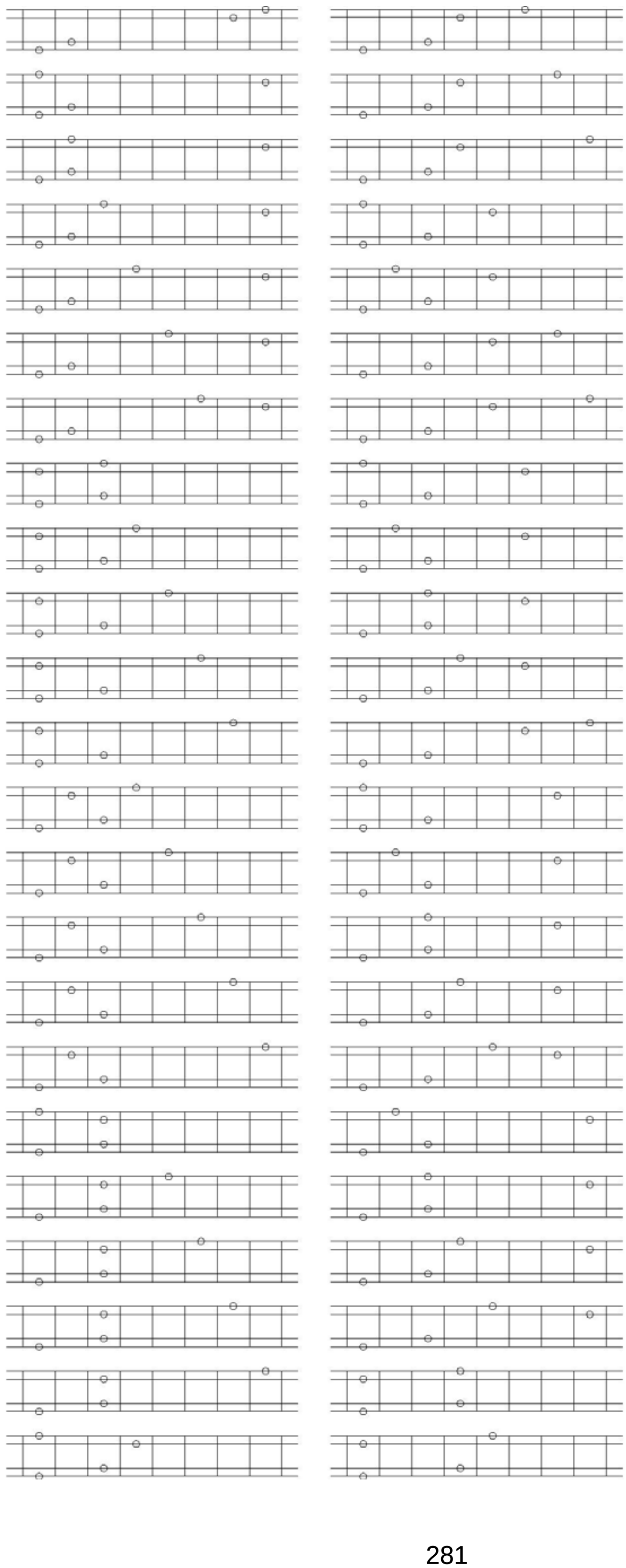

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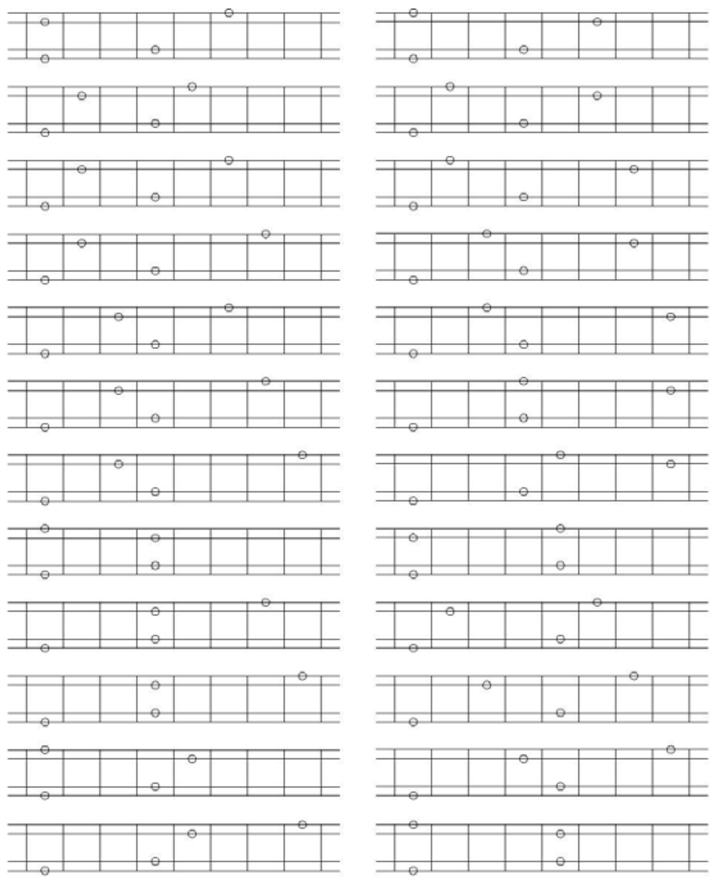
8-Unit Patterns – Page 3 of 3



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1	4	3	6
1	4	3	7
1	4	3	8
1	4	4	1
1	4	4	7
1	4	4	8
1	4	5	1
1	4	5	8

1	4	6	1
1	4	6	2
1	4	7	2
1	4	7	3
1	4	8	3
1	4	8	4
1	4	8	5
1	5	1	5
1	5	2	6
1	5	3	7
1	5	4	8
1	5	5	1

E.2 - Pattern Results

4-Unit Patterns - Sorted by Pattern

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c	C_1	C_2	C_3	R^2
4	1 2 1 2	0 3	1 1	2 0		0.7315	-0.8060	0.8238	0.9971
4	1 2 1 3	0 4	1 2	2 0		0.7760	-0.7873	0.8062	0.9966
4	1 2 1 4	0 3	1 1	2 0		0.7506	-0.8102	0.8277	0.9972
4	1 2 2 1	0 2	1 1	2 0		0.7017	-0.8305	0.8466	0.9977
4	1 2 2 3	0 2	1 1	2 0		0.7052	-0.8301	0.8461	0.9977
4	1 2 2 4	0 3	1 1	2 0		0.7455	-0.8075	0.8252	0.9971
4	1 2 3 1	0 4	1 2	2 0		0.7900	-0.7909	0.8096	0.9967
4	1 2 3 4	0 3	1 2	2 1	3 0	0.7699	-0.8147	0.8319	0.9973
4	1 2 4 1	0 4	1 2	2 0		0.7616	-0.8026	0.8205	0.9970
4	1 2 4 2	0 5	1 1	2 0		0.7815	-0.7795	0.7989	0.9964
4	1 2 4 3	0 4	1 2	2 1	3 0	0.7654	-0.8031	0.8209	0.9971
4	1 3 1 3	0 6	1 2	2 0		0.8052	-0.7610	0.7818	0.9957
4	1 3 2 4	0 5	1 3	2 1	3 0	0.8175	-0.7716	0.7917	0.9961
4	1 3 3 1	0 4	1 2	2 0		0.7869	-0.7899	0.8089	0.9966

4-Unit Patterns - Sorted by Strength

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c	C_1	C_2	C_3	R^2
4	1 3 1 3	0 6	1 2	2 0		0.8052	-0.7610	0.7818	0.9957
4	1 3 2 4	0 5	1 3	2 1	3 0	0.8175	-0.7716	0.7917	0.9961
4	1 2 4 2	0 5	1 1	2 0		0.7815	-0.7795	0.7989	0.9964
4	1 2 4 3	0 4	1 2	2 1	3 0	0.7654	-0.8031	0.8209	0.9971
4	1 2 1 3	0 4	1 2	2 0		0.7760	-0.7873	0.8062	0.9966
4	1 2 3 1	0 4	1 2	2 0		0.7900	-0.7909	0.8096	0.9967
4	1 2 4 1	0 4	1 2	2 0		0.7616	-0.8026	0.8205	0.9970
4	1 3 3 1	0 4	1 2	2 0		0.7869	-0.7899	0.8089	0.9966
4	1 2 3 4	0 3	1 2	2 1	3 0	0.7699	-0.8147	0.8319	0.9973
4	1 2 1 2	0 3	1 1	2 0		0.7315	-0.8060	0.8238	0.9971
4	1 2 1 4	0 3	1 1	2 0		0.7506	-0.8102	0.8277	0.9972
4	1 2 2 4	0 3	1 1	2 0		0.7455	-0.8075	0.8252	0.9971
4	1 2 2 1	0 2	1 1	2 0		0.7017	-0.8305	0.8466	0.9977
4	1 2 2 3	0 2	1 1	2 0		0.7052	-0.8301	0.8461	0.9977

5-Unit Patterns - Sorted by Pattern

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c	C_1	C_2	C_3	R^2
5	1 2 1 2	0 3	1 1	2 0		0.8170	-0.7647	0.7847	0.9960
5	1 2 1 3	0 4	1 2	2 0		0.8586	-0.7287	0.7503	0.9950
5	1 2 1 4	0 4	1 2	2 0		0.8685	-0.7306	0.7522	0.9951
5	1 2 1 5	0 3	1 1	2 0		0.8427	-0.7694	0.7892	0.9962
5	1 2 2 1	0 2	1 1	2 0		0.7939	-0.7922	0.8107	0.9968
5	1 2 2 3	0 2	1 1	2 0		0.7998	-0.7917	0.8101	0.9968
5	1 2 2 4	0 3	1 1	2 0		0.8400	-0.7505	0.7711	0.9957
5	1 2 2 5	0 3	1 1	2 0		0.8374	-0.7505	0.7712	0.9957
5	1 2 3 1	0 4	1 2	2 0		0.8751	-0.7325	0.7541	0.9951
5	1 2 3 4	0 3	1 2	2 1	3 0	0.8724	-0.7753	0.7947	0.9964
5	1 2 3 5	0 4	1 2	2 1	3 0	0.8875	-0.7344	0.7558	0.9952
5	1 2 4 1	0 5	1 2	2 0		0.8782	-0.7194	0.7415	0.9947
5	1 2 4 2	0 5	1 1	2 0		0.8691	-0.7174	0.7396	0.9946
5	1 2 4 3	0 4	1 2	2 1	3 0	0.8646	-0.7577	0.7778	0.9959
5	1 2 4 5	0 4	1 3	2 1	3 0	0.8724	-0.7595	0.7796	0.9960
5	1 2 5 1	0 4	1 2	2 0		0.8466	-0.7542	0.7746	0.9958
5	1 2 5 2	0 5	1 1	2 0		0.8599	-0.7157	0.7380	0.9946
5	1 2 5 3	0 5	1 2	2 1	3 0	0.8743	-0.7182	0.7402	0.9947
5	1 3 1 3	0 6	1 2	2 0		0.8829	-0.6866	0.7106	0.9933
5	1 3 1 4	0 6	1 2	2 0		0.8922	-0.6881	0.7119	0.9934
5	1 3 2 4	0 5	1 3	2 1	3 0	0.8987	-0.7005	0.7238	0.9939
5	1 3 2 5	0 5	1 3	2 1	3 0	0.9049	-0.7020	0.7252	0.9939
5	1 3 3 1	0 4	1 2	2 0		0.8799	-0.7202	0.7428	0.9945
5	1 3 3 5	0 4	1 2	2 0		0.8814	-0.7197	0.7423	0.9945
5	1 3 4 1	0 5	1 3	2 0		0.9116	-0.7037	0.7269	0.9939
5	1 3 5 2	0 6	1 3	2 1	3 0	0.8992	-0.6893	0.7131	0.9935

5-Unit Patterns - Sorted by Strength

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c	C_1	C_2	C_3	R^2
5	1 3 5 2	0 6	1 3	2 1	3 0	0.8992	-0.6893	0.7131	0.9935
5	1 3 1 3	0 6	1 2	2 0		0.8829	-0.6866	0.7106	0.9933
5	1 3 1 4	0 6	1 2	2 0		0.8922	-0.6881	0.7119	0.9934
5	1 3 2 4	0 5	1 3	2 1	3 0	0.8987	-0.7005	0.7238	0.9939
5	1 3 2 5	0 5	1 3	2 1	3 0	0.9049	-0.7020	0.7252	0.9939
5	1 3 4 1	0 5	1 3	2 0		0.9116	-0.7037	0.7269	0.9939
5	1 2 5 3	0 5	1 2	2 1	3 0	0.8743	-0.7182	0.7402	0.9947
5	1 2 4 1	0 5	1 2	2 0		0.8782	-0.7194	0.7415	0.9947
5	1 2 4 2	0 5	1 1	2 0		0.8691	-0.7174	0.7396	0.9946
5	1 2 5 2	0 5	1 1	2 0		0.8599	-0.7157	0.7380	0.9946
5	1 2 4 5	0 4	1 3	2 1	3 0	0.8724	-0.7595	0.7796	0.9960
5	1 2 3 5	0 4	1 2	2 1	3 0	0.8875	-0.7344	0.7558	0.9952
5	1 2 4 3	0 4	1 2	2 1	3 0	0.8646	-0.7577	0.7778	0.9959
5	1 2 1 3	0 4	1 2	2 0		0.8586	-0.7287	0.7503	0.9950
5	1 2 1 4	0 4	1 2	2 0		0.8685	-0.7306	0.7522	0.9951
5	1 2 3 1	0 4	1 2	2 0		0.8751	-0.7325	0.7541	0.9951
5	1 2 5 1	0 4	1 2	2 0		0.8466	-0.7542	0.7746	0.9958
5	1 3 3 1	0 4	1 2	2 0		0.8799	-0.7202	0.7428	0.9945
5	1 3 3 5	0 4	1 2	2 0		0.8814	-0.7197	0.7423	0.9945
5	1 2 3 4	0 3	1 2	2 1	3 0	0.8724	-0.7753	0.7947	0.9964
5	1 2 1 2	0 3	1 1	2 0		0.8170	-0.7647	0.7847	0.9960
5	1 2 1 5	0 3	1 1	2 0		0.8427	-0.7694	0.7892	0.9962
5	1 2 2 4	0 3	1 1	2 0		0.8400	-0.7505	0.7711	0.9957
5	1 2 2 5	0 3	1 1	2 0		0.8374	-0.7505	0.7712	0.9957
5	1 2 2 1	0 2	1 1	2 0		0.7939	-0.7922	0.8107	0.9968
5	1 2 2 3	0 2	1 1	2 0		0.7998	-0.7917	0.8101	0.9968

6-Unit Patterns - Sorted by Pattern

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c	C_1	C_2	C_3	R^2
6	1 2 1 2	0 3	1 1	2 0		0.8771	-0.7318	0.7533	0.9951
6	1 2 1 3	0 4	1 2	2 0		0.9080	-0.6841	0.7072	0.9937
6	1 2 1 4	0 5	1 2	2 0		0.9160	-0.6693	0.6927	0.9933
6	1 2 1 5	0 4	1 2	2 0		0.9241	-0.6870	0.7101	0.9938
6	1 2 1 6	0 3	1 1	2 0		0.9061	-0.7368	0.7581	0.9953
6	1 2 2 1	0 2	1 1	2 0		0.8592	-0.7613	0.7814	0.9960
6	1 2 2 3	0 2	1 1	2 0		0.8671	-0.7610	0.7809	0.9960
6	1 2 2 4	0 3	1 1	2 0		0.8976	-0.7069	0.7291	0.9945
6	1 2 2 5	0 4	1 1	2 0		0.8951	-0.6887	0.7114	0.9940
6	1 2 2 6	0 3	1 1	2 0		0.8921	-0.7066	0.7289	0.9944
6	1 2 3 1	0 4	1 2	2 0		0.9249	-0.6879	0.7109	0.9938
6	1 2 3 4	0 3	1 2	2 1	3 0	0.9433	-0.7437	0.7647	0.9955
6	1 2 3 5	0 4	1 2	2 1	3 0	0.9480	-0.6914	0.7143	0.9940
6	1 2 3 6	0 5	1 2	2 1	3 0	0.9328	-0.6726	0.6959	0.9934
6	1 2 4 1	0 6	1 2	2 0		0.9229	-0.6564	0.6802	0.9929
6	1 2 4 2	0 5	1 1	2 0		0.9196	-0.6706	0.6942	0.9933
6	1 2 4 3	0 4	1 2	2 1	3 0	0.9308	-0.7220	0.7436	0.9949
6	1 2 4 5	0 4	1 3	2 1	3 0	0.9498	-0.7257	0.7473	0.9950
6	1 2 4 6	0 5	1 3	2 1	3 0	0.9430	-0.6750	0.6984	0.9934
6	1 2 5 1	0 6	1 2	2 0		0.9224	-0.6660	0.6896	0.9932
6	1 2 5 2	0 7	1 1	2 0		0.9088	-0.6485	0.6725	0.9926
6	1 2 5 3	0 6	1 2	2 1	3 0	0.9293	-0.6670	0.6905	0.9933
6	1 2 5 4	0 5	1 3	2 1	3 0	0.9389	-0.7172	0.7389	0.9949
6	1 2 5 6	0 5	1 3	2 1	3 0	0.9317	-0.7159	0.7377	0.9948
6	1 2 6 1	0 4	1 2	2 0		0.9031	-0.7167	0.7385	0.9948
6	1 2 6 2	0 5	1 1	2 0		0.9049	-0.6680	0.6916	0.9932
6	1 2 6 3	0 6	1 2	2 1	3 0	0.9152	-0.6544	0.6781	0.9929
6	1 2 6 4	0 5	1 3	2 1	3 0	0.9337	-0.6731	0.6963	0.9935
6	1 3 1 3	0 6	1 2	2 0		0.9205	-0.6318	0.6571	0.9915
6	1 3 1 4	0 7	1 3	2 0		0.9288	-0.6204	0.6459	0.9911
6	1 3 1 5	0 6	1 2	2 0		0.9385	-0.6347	0.6598	0.9917
6	1 3 2 4	0 5	1 3	2 1	3 0	0.9396	-0.6477	0.6725	0.9922
6	1 3 2 5	0 6	1 3	2 1	3 0	0.9446	-0.6355	0.6606	0.9917
6	1 3 2 6	0 5	1 3	2 1	3 0	0.9482	-0.6496	0.6745	0.9922
6	1 3 3 1	0 4	1 2	2 0		0.9254	-0.6674	0.6919	0.9927
6	1 3 3 5	0 4	1 2	2 0		0.9302	-0.6671	0.6914	0.9928
6	1 3 3 6	0 5	1 2	2 0		0.9282	-0.6527	0.6776	0.9922
6	1 3 4 1	0 6	1 3	2 0		0.9524	-0.6376	0.6628	0.9916
6	1 3 4 6	0 5	1 3	2 1	3 0	0.9617	-0.6520	0.6768	0.9922
6	1 3 5 1	0 6	1 4	2 0		0.9517	-0.6374	0.6625	0.9918
6	1 3 5 2	0 7	1 3	2 1	3 0	0.9415	-0.6227	0.6481	0.9912
6	1 3 6 2	0 7	1 3	2 1	3 0	0.9335	-0.6293	0.6545	0.9915
6	1 3 6 3	0 8	1 2	2 0		0.9258	-0.6157	0.6413	0.9909
6	1 3 6 4	0 7	1 3	2 1	3 0	0.9384	-0.6301	0.6551	0.9916
6	1 4 1 4	0 9	1 3	2 0		0.9180	-0.6028	0.6288	0.9903
6	1 4 2 5	0 8	1 4	2 1	3 0	0.9342	-0.6094	0.6352	0.9906
6	1 4 3 6	0 7	1 4	2 1	3 0	0.9481	-0.6239	0.6495	0.9911
6	1 4 4 1	0 6	1 3	2 0		0.9283	-0.6397	0.6651	0.9916

6-Unit Patterns - Sorted by Strength

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c	C_1	C_2	C_3	R^2
6	1 4 1 4	0 9	1 3	2 0		0.9180	-0.6028	0.6288	0.9903
6	1 4 2 5	0 8	1 4	2 1	3 0	0.9342	-0.6094	0.6352	0.9906
6	1 3 6 3	0 8	1 2	2 0		0.9258	-0.6157	0.6413	0.9909
6	1 4 3 6	0 7	1 4	2 1	3 0	0.9481	-0.6239	0.6495	0.9911
6	1 3 5 2	0 7	1 3	2 1	3 0	0.9415	-0.6227	0.6481	0.9912
6	1 3 6 2	0 7	1 3	2 1	3 0	0.9335	-0.6293	0.6545	0.9915
6	1 3 6 4	0 7	1 3	2 1	3 0	0.9384	-0.6301	0.6551	0.9916
6	1 3 1 4	0 7	1 3	2 0		0.9288	-0.6204	0.6459	0.9911
6	1 2 5 2	0 7	1 1	2 0		0.9088	-0.6485	0.6725	0.9926
6	1 3 5 1	0 6	1 4	2 0		0.9517	-0.6374	0.6625	0.9918
6	1 3 2 5	0 6	1 3	2 1	3 0	0.9446	-0.6355	0.6606	0.9917
6	1 3 4 1	0 6	1 3	2 0		0.9524	-0.6376	0.6628	0.9916
6	1 4 4 1	0 6	1 3	2 0		0.9283	-0.6397	0.6651	0.9916
6	1 2 5 3	0 6	1 2	2 1	3 0	0.9293	-0.6670	0.6905	0.9933
6	1 2 6 3	0 6	1 2	2 1	3 0	0.9152	-0.6544	0.6781	0.9929
6	1 2 4 1	0 6	1 2	2 0		0.9229	-0.6564	0.6802	0.9929
6	1 2 5 1	0 6	1 2	2 0		0.9224	-0.6660	0.6896	0.9932
6	1 3 1 3	0 6	1 2	2 0		0.9205	-0.6318	0.6571	0.9915
6	1 3 1 5	0 6	1 2	2 0		0.9385	-0.6347	0.6598	0.9917
6	1 2 4 6	0 5	1 3	2 1	3 0	0.9430	-0.6750	0.6984	0.9934
6	1 2 5 4	0 5	1 3	2 1	3 0	0.9389	-0.7172	0.7389	0.9949
6	1 2 5 6	0 5	1 3	2 1	3 0	0.9317	-0.7159	0.7377	0.9948
6	1 2 6 4	0 5	1 3	2 1	3 0	0.9337	-0.6731	0.6963	0.9935
6	1 3 2 4	0 5	1 3	2 1	3 0	0.9396	-0.6477	0.6725	0.9922
6	1 3 2 6	0 5	1 3	2 1	3 0	0.9482	-0.6496	0.6745	0.9922
6	1 3 4 6	0 5	1 3	2 1	3 0	0.9617	-0.6520	0.6768	0.9922
6	1 2 3 6	0 5	1 2	2 1	3 0	0.9328	-0.6726	0.6959	0.9934
6	1 2 1 4	0 5	1 2	2 0		0.9160	-0.6693	0.6927	0.9933
6	1 3 3 6	0 5	1 2	2 0		0.9282	-0.6527	0.6776	0.9922
6	1 2 4 2	0 5	1 1	2 0		0.9196	-0.6706	0.6942	0.9933
6	1 2 6 2	0 5	1 1	2 0		0.9049	-0.6680	0.6916	0.9932
6	1 2 4 5	0 4	1 3	2 1	3 0	0.9498	-0.7257	0.7473	0.9950
6	1 2 3 5	0 4	1 2	2 1	3 0	0.9480	-0.6914	0.7143	0.9940
6	1 2 4 3	0 4	1 2	2 1	3 0	0.9308	-0.7220	0.7436	0.9949
6	1 2 1 3	0 4	1 2	2 0		0.9080	-0.6841	0.7072	0.9937
6	1 2 1 5	0 4	1 2	2 0		0.9241	-0.6870	0.7101	0.9938
6	1 2 3 1	0 4	1 2	2 0		0.9249	-0.6879	0.7109	0.9938
6	1 2 6 1	0 4	1 2	2 0		0.9031	-0.7167	0.7385	0.9948
6	1 3 3 1	0 4	1 2	2 0		0.9254	-0.6674	0.6919	0.9927
6	1 3 3 5	0 4	1 2	2 0		0.9302	-0.6671	0.6914	0.9928
6	1 2 2 5	0 4	1 1	2 0		0.8951	-0.6887	0.7114	0.9940
6	1 2 3 4	0 3	1 2	2 1	3 0	0.9433	-0.7437	0.7647	0.9955
6	1 2 1 2	0 3	1 1	2 0		0.8771	-0.7318	0.7533	0.9951
6	1 2 1 6	0 3	1 1	2 0		0.9061	-0.7368	0.7581	0.9953
6	1 2 2 4	0 3	1 1	2 0		0.8976	-0.7069	0.7291	0.9945
6	1 2 2 6	0 3	1 1	2 0		0.8921	-0.7066	0.7289	0.9944
6	1 2 2 1	0 2	1 1	2 0		0.8592	-0.7613	0.7814	0.9960
6	1 2 2 3	0 2	1 1	2 0		0.8671	-0.7610	0.7809	0.9960

7-Unit Patterns - Sorted by Pattern

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c	C_1	C_2	C_3	R^2
7	1 2 1 2	0 3	1 1	2 0		0.9227	-0.7014	0.7239	0.9943
7	1 2 1 3	0 4	1 2	2 0		0.9418	-0.6447	0.6685	0.9927
7	1 2 1 4	0 5	1 2	2 0		0.9422	-0.6182	0.6421	0.9920
7	1 2 1 5	0 5	1 2	2 0		0.9483	-0.6192	0.6431	0.9920
7	1 2 1 6	0 4	1 2	2 0		0.9608	-0.6477	0.6715	0.9927
7	1 2 1 7	0 3	1 1	2 0		0.9527	-0.7058	0.7282	0.9944
7	1 2 2 1	0 2	1 1	2 0		0.9091	-0.7321	0.7534	0.9952
7	1 2 2 3	0 2	1 1	2 0		0.9189	-0.7318	0.7529	0.9953
7	1 2 2 4	0 3	1 1	2 0		0.9372	-0.6679	0.6909	0.9935
7	1 2 2 5	0 4	1 1	2 0		0.9277	-0.6371	0.6605	0.9927
7	1 2 2 6	0 4	1 1	2 0		0.9247	-0.6368	0.6603	0.9927
7	1 2 2 7	0 3	1 1	2 0		0.9290	-0.6674	0.6906	0.9934
7	1 2 3 1	0 4	1 2	2 0		0.9577	-0.6480	0.6719	0.9927
7	1 2 3 4	0 3	1 2	2 1	3 0	0.9950	-0.7131	0.7352	0.9946
7	1 2 3 5	0 4	1 2	2 1	3 0	0.9887	-0.6524	0.6761	0.9929
7	1 2 3 6	0 5	1 2	2 1	3 0	0.9673	-0.6224	0.6463	0.9921
7	1 2 3 7	0 5	1 2	2 1	3 0	0.9563	-0.6207	0.6447	0.9921
7	1 2 4 1	0 6	1 2	2 0		0.9457	-0.6037	0.6280	0.9914
7	1 2 4 2	0 5	1 1	2 0		0.9522	-0.6291	0.6533	0.9921
7	1 2 4 3	0 4	1 2	2 1	3 0	0.9777	-0.6883	0.7110	0.9940
7	1 2 4 5	0 4	1 3	2 1	3 0	1.0049	-0.6932	0.7160	0.9940
7	1 2 4 6	0 5	1 3	2 1	3 0	0.9878	-0.6353	0.6594	0.9923
7	1 2 4 7	0 6	1 3	2 1	3 0	0.9600	-0.6060	0.6302	0.9916
7	1 2 5 1	0 7	1 2	2 0		0.9412	-0.5950	0.6193	0.9912
7	1 2 5 2	0 7	1 1	2 0		0.9335	-0.5934	0.6178	0.9911
7	1 2 5 3	0 6	1 2	2 1	3 0	0.9638	-0.6225	0.6466	0.9920
7	1 2 5 4	0 5	1 3	2 1	3 0	0.9897	-0.6800	0.7027	0.9938
7	1 2 5 6	0 5	1 4	2 1	3 0	0.9931	-0.6805	0.7034	0.9938
7	1 2 5 7	0 6	1 3	2 1	3 0	0.9708	-0.6239	0.6480	0.9920
7	1 2 6 1	0 6	1 2	2 0		0.9494	-0.6202	0.6443	0.9919
7	1 2 6 2	0 7	1 1	2 0		0.9282	-0.5926	0.6169	0.9911
7	1 2 6 3	0 7	1 2	2 1	3 0	0.9434	-0.5951	0.6192	0.9913
7	1 2 6 4	0 6	1 3	2 1	3 0	0.9734	-0.6243	0.6482	0.9922
7	1 2 6 7	0 5	1 3	2 1	3 0	0.9711	-0.6766	0.6994	0.9937
7	1 2 7 1	0 4	1 2	2 0		0.9442	-0.6825	0.7052	0.9939
7	1 2 7 2	0 5	1 1	2 0		0.9349	-0.6264	0.6506	0.9921
7	1 2 7 3	0 6	1 2	2 1	3 0	0.9371	-0.6017	0.6258	0.9915
7	1 2 7 4	0 6	1 3	2 1	3 0	0.9517	-0.6043	0.6283	0.9917
7	1 2 7 5	0 5	1 3	2 1	3 0	0.9735	-0.6328	0.6567	0.9924
7	1 3 1 3	0 6	1 2	2 0		0.9421	-0.5850	0.6106	0.9900
7	1 3 1 4	0 7	1 3	2 0		0.9442	-0.5645	0.5903	0.9892
7	1 3 1 5	0 7	1 3	2 0		0.9529	-0.5659	0.5917	0.9893
7	1 3 1 6	0 6	1 2	2 0		0.9649	-0.5882	0.6137	0.9902
7	1 3 2 4	0 5	1 3	2 1	3 0	0.9634	-0.6019	0.6271	0.9907
7	1 3 2 5	0 6	1 3	2 1	3 0	0.9620	-0.5801	0.6056	0.9899
7	1 3 2 6	0 6	1 3	2 1	3 0	0.9644	-0.5805	0.6062	0.9898
7	1 3 2 7	0 5	1 3	2 1	3 0	0.9719	-0.6036	0.6291	0.9906
7	1 3 3 1	0 4	1 2	2 0		0.9509	-0.6211	0.6464	0.9912
7	1 3 3 5	0 4	1 2	2 0		0.9592	-0.6210	0.6460	0.9914
7	1 3 3 6	0 5	1 2	2 0		0.9507	-0.5964	0.6219	0.9904
7	1 3 3 7	0 5	1 2	2 0		0.9472	-0.5962	0.6219	0.9903

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c	C_1	C_2	C_3	R^2
7	1 3 4 1	0 6	1 3	2 0		0.9679	-0.5818	0.6076	0.9898
7	1 3 4 6	0 5	1 3	2 1	3 0	0.9916	-0.6065	0.6319	0.9908
7	1 3 4 7	0 6	1 3	2 1	3 0	0.9750	-0.5824	0.6082	0.9898
7	1 3 5 1	0 7	1 4	2 0		0.9660	-0.5684	0.5943	0.9893
7	1 3 5 2	0 7	1 3	2 1	3 0	0.9590	-0.5669	0.5927	0.9893
7	1 3 5 7	0 6	1 4	2 1	3 0	0.9846	-0.5915	0.6171	0.9902
7	1 3 6 1	0 7	1 4	2 0		0.9678	-0.5818	0.6073	0.9900
7	1 3 6 2	0 8	1 3	2 1	3 0	0.9509	-0.5590	0.5847	0.9891
7	1 3 6 3	0 8	1 2	2 0		0.9466	-0.5583	0.5840	0.9890
7	1 3 6 4	0 7	1 3	2 1	3 0	0.9677	-0.5814	0.6068	0.9901
7	1 3 7 2	0 7	1 3	2 1	3 0	0.9512	-0.5788	0.6043	0.9899
7	1 3 7 3	0 8	1 2	2 0		0.9382	-0.5569	0.5827	0.9890
7	1 3 7 4	0 8	1 3	2 1	3 0	0.9489	-0.5585	0.5841	0.9892
7	1 4 1 4	0 9	1 3	2 0		0.9266	-0.5370	0.5632	0.9879
7	1 4 1 5	0 9	1 3	2 0		0.9338	-0.5380	0.5641	0.9880
7	1 4 2 5	0 8	1 4	2 1	3 0	0.9440	-0.5456	0.5716	0.9884
7	1 4 2 6	0 8	1 4	2 1	3 0	0.9497	-0.5466	0.5726	0.9884
7	1 4 3 6	0 7	1 4	2 1	3 0	0.9589	-0.5599	0.5860	0.9889
7	1 4 3 7	0 7	1 4	2 1	3 0	0.9600	-0.5603	0.5865	0.9888
7	1 4 4 1	0 6	1 3	2 0		0.9413	-0.5744	0.6006	0.9892
7	1 4 4 7	0 6	1 3	2 0		0.9431	-0.5741	0.6003	0.9893
7	1 4 5 1	0 7	1 4	2 0		0.9627	-0.5611	0.5873	0.9888
7	1 4 6 2	0 8	1 5	2 1	3 0	0.9545	-0.5476	0.5737	0.9884
7	1 4 7 3	0 9	1 4	2 1	3 0	0.9388	-0.5387	0.5647	0.9881

7-Unit Patterns - Sorted by Strength

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c	C_1	C_2	C_3	R^2
7	1 4 7 3	0 9	1 4	2 1	3 0	0.9388	-0.5387	0.5647	0.9881
7	1 4 1 4	0 9	1 3	2 0		0.9266	-0.5370	0.5632	0.9879
7	1 4 1 5	0 9	1 3	2 0		0.9338	-0.5380	0.5641	0.9880
7	1 4 6 2	0 8	1 5	2 1	3 0	0.9545	-0.5476	0.5737	0.9884
7	1 4 2 5	0 8	1 4	2 1	3 0	0.9440	-0.5456	0.5716	0.9884
7	1 4 2 6	0 8	1 4	2 1	3 0	0.9497	-0.5466	0.5726	0.9884
7	1 3 6 2	0 8	1 3	2 1	3 0	0.9509	-0.5590	0.5847	0.9891
7	1 3 7 4	0 8	1 3	2 1	3 0	0.9489	-0.5585	0.5841	0.9892
7	1 3 6 3	0 8	1 2	2 0		0.9466	-0.5583	0.5840	0.9890
7	1 3 7 3	0 8	1 2	2 0		0.9382	-0.5569	0.5827	0.9890
7	1 4 3 6	0 7	1 4	2 1	3 0	0.9589	-0.5599	0.5860	0.9889
7	1 4 3 7	0 7	1 4	2 1	3 0	0.9600	-0.5603	0.5865	0.9888
7	1 3 5 1	0 7	1 4	2 0		0.9660	-0.5684	0.5943	0.9893
7	1 3 6 1	0 7	1 4	2 0		0.9678	-0.5818	0.6073	0.9900
7	1 4 5 1	0 7	1 4	2 0		0.9627	-0.5611	0.5873	0.9888
7	1 3 5 2	0 7	1 3	2 1	3 0	0.9590	-0.5669	0.5927	0.9893
7	1 3 6 4	0 7	1 3	2 1	3 0	0.9677	-0.5814	0.6068	0.9901
7	1 3 7 2	0 7	1 3	2 1	3 0	0.9512	-0.5788	0.6043	0.9899
7	1 3 1 4	0 7	1 3	2 0		0.9442	-0.5645	0.5903	0.9892
7	1 3 1 5	0 7	1 3	2 0		0.9529	-0.5659	0.5917	0.9893
7	1 2 6 3	0 7	1 2	2 1	3 0	0.9434	-0.5951	0.6192	0.9913

N	Pattern				F_m	F_c	F_m	F_c	F_m	F_c	F_m	F_c	C_1	C_2	C_3	R^2
7	1	2	5	1	0	7	1	2	2	0			0.9412	-0.5950	0.6193	0.9912
7	1	2	5	2	0	7	1	1	2	0			0.9335	-0.5934	0.6178	0.9911
7	1	2	6	2	0	7	1	1	2	0			0.9282	-0.5926	0.6169	0.9911
7	1	3	5	7	0	6	1	4	2	1	3	0	0.9846	-0.5915	0.6171	0.9902
7	1	2	4	7	0	6	1	3	2	1	3	0	0.9600	-0.6060	0.6302	0.9916
7	1	2	5	7	0	6	1	3	2	1	3	0	0.9708	-0.6239	0.6480	0.9920
7	1	2	6	4	0	6	1	3	2	1	3	0	0.9734	-0.6243	0.6482	0.9922
7	1	2	7	4	0	6	1	3	2	1	3	0	0.9517	-0.6043	0.6283	0.9917
7	1	3	2	5	0	6	1	3	2	1	3	0	0.9620	-0.5801	0.6056	0.9899
7	1	3	2	6	0	6	1	3	2	1	3	0	0.9644	-0.5805	0.6062	0.9898
7	1	3	4	7	0	6	1	3	2	1	3	0	0.9750	-0.5824	0.6082	0.9898
7	1	3	4	1	0	6	1	3	2	0			0.9679	-0.5818	0.6076	0.9898
7	1	4	4	1	0	6	1	3	2	0			0.9413	-0.5744	0.6006	0.9892
7	1	4	4	7	0	6	1	3	2	0			0.9431	-0.5741	0.6003	0.9893
7	1	2	5	3	0	6	1	2	2	1	3	0	0.9638	-0.6225	0.6466	0.9920
7	1	2	7	3	0	6	1	2	2	1	3	0	0.9371	-0.6017	0.6258	0.9915
7	1	2	4	1	0	6	1	2	2	0			0.9457	-0.6037	0.6280	0.9914
7	1	2	6	1	0	6	1	2	2	0			0.9494	-0.6202	0.6443	0.9919
7	1	3	1	3	0	6	1	2	2	0			0.9421	-0.5850	0.6106	0.9900
7	1	3	1	6	0	6	1	2	2	0			0.9649	-0.5882	0.6137	0.9902
7	1	2	5	6	0	5	1	4	2	1	3	0	0.9931	-0.6805	0.7034	0.9938
7	1	2	4	6	0	5	1	3	2	1	3	0	0.9878	-0.6353	0.6594	0.9923
7	1	2	5	4	0	5	1	3	2	1	3	0	0.9897	-0.6800	0.7027	0.9938
7	1	2	6	7	0	5	1	3	2	1	3	0	0.9711	-0.6766	0.6994	0.9937
7	1	2	7	5	0	5	1	3	2	1	3	0	0.9735	-0.6328	0.6567	0.9924
7	1	3	2	4	0	5	1	3	2	1	3	0	0.9634	-0.6019	0.6271	0.9907
7	1	3	2	7	0	5	1	3	2	1	3	0	0.9719	-0.6036	0.6291	0.9906
7	1	3	4	6	0	5	1	3	2	1	3	0	0.9916	-0.6065	0.6319	0.9908
7	1	2	3	6	0	5	1	2	2	1	3	0	0.9673	-0.6224	0.6463	0.9921
7	1	2	3	7	0	5	1	2	2	1	3	0	0.9563	-0.6207	0.6447	0.9921
7	1	2	1	4	0	5	1	2	2	0			0.9422	-0.6182	0.6421	0.9920
7	1	2	1	5	0	5	1	2	2	0			0.9483	-0.6192	0.6431	0.9920
7	1	3	3	6	0	5	1	2	2	0			0.9507	-0.5964	0.6219	0.9904
7	1	3	3	7	0	5	1	2	2	0			0.9472	-0.5962	0.6219	0.9903
7	1	2	4	2	0	5	1	1	2	0			0.9522	-0.6291	0.6533	0.9921
7	1	2	7	2	0	5	1	1	2	0			0.9349	-0.6264	0.6506	0.9921
7	1	2	4	5	0	4	1	3	2	1	3	0	1.0049	-0.6932	0.7160	0.9940
7	1	2	3	5	0	4	1	2	2	1	3	0	0.9887	-0.6524	0.6761	0.9929
7	1	2	4	3	0	4	1	2	2	1	3	0	0.9777	-0.6883	0.7110	0.9940
7	1	2	1	3	0	4	1	2	2	0			0.9418	-0.6447	0.6685	0.9927
7	1	2	1	6	0	4	1	2	2	0			0.9608	-0.6477	0.6715	0.9927
7	1	2	3	1	0	4	1	2	2	0			0.9577	-0.6480	0.6719	0.9927
7	1	2	7	1	0	4	1	2	2	0			0.9442	-0.6825	0.7052	0.9939
7	1	3	3	1	0	4	1	2	2	0			0.9509	-0.6211	0.6464	0.9912
7	1	3	3	5	0	4	1	2	2	0			0.9592	-0.6210	0.6460	0.9914
7	1	2	2	5	0	4	1	1	2	0			0.9277	-0.6371	0.6605	0.9927
7	1	2	2	6	0	4	1	1	2	0			0.9247	-0.6368	0.6603	0.9927
7	1	2	3	4	0	3	1	2	2	1	3	0	0.9950	-0.7131	0.7352	0.9946
7	1	2	1	2	0	3	1	1	2	0			0.9227	-0.7014	0.7239	0.9943
7	1	2	1	7	0	3	1	1	2	0			0.9527	-0.7058	0.7282	0.9944
7	1	2	2	4	0	3	1	1	2	0			0.9372	-0.6679	0.6909	0.9935
7	1	2	2	7	0	3	1	1	2	0			0.9290	-0.6674	0.6906	0.9934
7	1	2	2	1	0	2	1	1	2	0			0.9091	-0.7321	0.7534	0.9952
7	1	2	2	3	0	2	1	1	2	0			0.9189	-0.7318	0.7529	0.9953

8-Unit Patterns - Sorted by Pattern

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c	C_1	C_2	C_3	R^2
8	1 2 1 2	0 3	1 1	2 0		0.9587	-0.6760	0.6991	0.9936
8	1 2 1 3	0 4	1 2	2 0		0.9684	-0.6135	0.6375	0.9919
8	1 2 1 4	0 5	1 2	2 0		0.9621	-0.5795	0.6033	0.9911
8	1 2 1 5	0 6	1 2	2 0		0.9620	-0.5692	0.5929	0.9909
8	1 2 1 6	0 5	1 2	2 0		0.9711	-0.5807	0.6047	0.9911
8	1 2 1 7	0 4	1 2	2 0		0.9880	-0.6162	0.6404	0.9918
8	1 2 1 8	0 3	1 1	2 0		0.9885	-0.6799	0.7031	0.9936
8	1 2 2 1	0 2	1 1	2 0		0.9485	-0.7076	0.7297	0.9945
8	1 2 2 3	0 2	1 1	2 0		0.9596	-0.7073	0.7292	0.9946
8	1 2 2 4	0 3	1 1	2 0		0.9677	-0.6368	0.6603	0.9927
8	1 2 2 5	0 4	1 1	2 0		0.9519	-0.5979	0.6214	0.9918
8	1 2 2 6	0 5	1 1	2 0		0.9434	-0.5854	0.6089	0.9915
8	1 2 2 7	0 4	1 1	2 0		0.9461	-0.5973	0.6209	0.9917
8	1 2 2 8	0 3	1 1	2 0		0.9575	-0.6361	0.6598	0.9926
8	1 2 3 1	0 4	1 2	2 0		0.9829	-0.6163	0.6406	0.9918
8	1 2 3 4	0 3	1 2	2 1	3 0	1.0333	-0.6872	0.7102	0.9938
8	1 2 3 5	0 4	1 2	2 1	3 0	1.0186	-0.6211	0.6452	0.9920
8	1 2 3 6	0 5	1 2	2 1	3 0	0.9928	-0.5842	0.6081	0.9912
8	1 2 3 7	0 6	1 2	2 1	3 0	0.9771	-0.5715	0.5953	0.9909
8	1 2 3 8	0 5	1 2	2 1	3 0	0.9737	-0.5813	0.6053	0.9911
8	1 2 4 1	0 6	1 2	2 0		0.9628	-0.5640	0.5882	0.9904
8	1 2 4 2	0 5	1 1	2 0		0.9768	-0.5964	0.6208	0.9911
8	1 2 4 3	0 4	1 2	2 1	3 0	1.0120	-0.6604	0.6838	0.9932
8	1 2 4 5	0 4	1 3	2 1	3 0	1.0442	-0.6658	0.6895	0.9932
8	1 2 4 6	0 5	1 3	2 1	3 0	1.0205	-0.6036	0.6281	0.9914
8	1 2 4 7	0 6	1 3	2 1	3 0	0.9893	-0.5681	0.5922	0.9906
8	1 2 4 8	0 7	1 3	2 1	3 0	0.9688	-0.5553	0.5792	0.9903
8	1 2 5 1	0 8	1 2	2 0		0.9512	-0.5433	0.5673	0.9899
8	1 2 5 2	0 7	1 1	2 0		0.9515	-0.5522	0.5763	0.9901
8	1 2 5 3	0 6	1 2	2 1	3 0	0.9885	-0.5876	0.6118	0.9911
8	1 2 5 4	0 5	1 3	2 1	3 0	1.0251	-0.6492	0.6727	0.9929
8	1 2 5 6	0 5	1 4	2 1	3 0	1.0366	-0.6511	0.6748	0.9928
8	1 2 5 7	0 6	1 4	2 1	3 0	1.0079	-0.5910	0.6154	0.9911
8	1 2 5 8	0 7	1 3	2 1	3 0	0.9730	-0.5560	0.5801	0.9903
8	1 2 6 1	0 8	1 2	2 0		0.9544	-0.5498	0.5738	0.9901
8	1 2 6 2	0 9	1 1	2 0		0.9401	-0.5382	0.5621	0.9898
8	1 2 6 3	0 8	1 2	2 1	3 0	0.9627	-0.5510	0.5749	0.9902
8	1 2 6 4	0 7	1 3	2 1	3 0	1.0003	-0.5863	0.6104	0.9911
8	1 2 6 5	0 6	1 4	2 1	3 0	1.0292	-0.6456	0.6691	0.9929
8	1 2 6 7	0 6	1 4	2 1	3 0	1.0210	-0.6441	0.6677	0.9928
8	1 2 6 8	0 7	1 3	2 1	3 0	0.9897	-0.5845	0.6087	0.9910
8	1 2 7 1	0 6	1 2	2 0		0.9705	-0.5848	0.6090	0.9910
8	1 2 7 2	0 7	1 1	2 0		0.9437	-0.5511	0.5752	0.9901
8	1 2 7 3	0 8	1 2	2 1	3 0	0.9514	-0.5432	0.5668	0.9901
8	1 2 7 4	0 7	1 3	2 1	3 0	0.9746	-0.5564	0.5802	0.9905
8	1 2 7 5	0 6	1 4	2 1	3 0	1.0055	-0.5908	0.6149	0.9912
8	1 2 7 8	0 5	1 3	2 1	3 0	1.0002	-0.6449	0.6683	0.9929
8	1 2 8 1	0 4	1 2	2 0		0.9763	-0.6545	0.6779	0.9931
8	1 2 8 2	0 5	1 1	2 0		0.9589	-0.5939	0.6182	0.9912
8	1 2 8 3	0 6	1 2	2 1	3 0	0.9544	-0.5622	0.5861	0.9906
8	1 2 8 4	0 7	1 3	2 1	3 0	0.9620	-0.5541	0.5778	0.9905
8	1 2 8 5	0 6	1 3	2 1	3 0	0.9785	-0.5664	0.5903	0.9907
8	1 2 8 6	0 5	1 3	2 1	3 0	1.0020	-0.6006	0.6249	0.9914

N	Pattern				F_m	F_c	F_m	F_c	F_m	F_c	F_m	F_c	C_1	C_2	C_3	R^2
8	1	3	1	3	0	6	1	2	2	0			0.9591	-0.5489	0.5744	0.9889
8	1	3	1	4	0	7	1	3	2	0			0.9558	-0.5227	0.5481	0.9879
8	1	3	1	5	0	8	1	4	2	0			0.9594	-0.5152	0.5405	0.9877
8	1	3	1	6	0	7	1	3	2	0			0.9695	-0.5247	0.5502	0.9880
8	1	3	1	7	0	6	1	2	2	0			0.9835	-0.5519	0.5775	0.9890
8	1	3	2	4	0	5	1	3	2	1	3	0	0.9817	-0.5662	0.5915	0.9897
8	1	3	2	5	0	6	1	3	2	1	3	0	0.9747	-0.5383	0.5637	0.9886
8	1	3	2	6	0	7	1	3	2	1	3	0	0.9719	-0.5294	0.5548	0.9883
8	1	3	2	7	0	6	1	3	2	1	3	0	0.9771	-0.5386	0.5642	0.9885
8	1	3	2	8	0	5	1	3	2	1	3	0	0.9890	-0.5675	0.5931	0.9895
8	1	3	3	1	0	4	1	2	2	0			0.9695	-0.5849	0.6105	0.9901
8	1	3	3	5	0	4	1	2	2	0			0.9808	-0.5851	0.6103	0.9903
8	1	3	3	6	0	5	1	2	2	0			0.9667	-0.5538	0.5793	0.9891
8	1	3	3	7	0	6	1	2	2	0			0.9580	-0.5435	0.5690	0.9887
8	1	3	3	8	0	5	1	2	2	0			0.9595	-0.5533	0.5790	0.9889
8	1	3	4	1	0	6	1	3	2	0			0.9782	-0.5394	0.5652	0.9883
8	1	3	4	6	0	5	1	3	2	1	3	0	1.0134	-0.5709	0.5965	0.9896
8	1	3	4	7	0	6	1	3	2	1	3	0	0.9918	-0.5408	0.5665	0.9885
8	1	3	4	8	0	7	1	3	2	1	3	0	0.9787	-0.5305	0.5562	0.9881
8	1	3	5	1	0	8	1	4	2	0			0.9708	-0.5171	0.5427	0.9876
8	1	3	5	2	0	7	1	3	2	1	3	0	0.9706	-0.5247	0.5503	0.9879
8	1	3	5	7	0	6	1	4	2	2	3	0	1.0092	-0.5558	0.5815	0.9891
8	1	3	5	8	0	7	1	4	2	1	3	0	0.9850	-0.5270	0.5526	0.9879
8	1	3	6	1	0	8	1	4	2	0			0.9710	-0.5172	0.5427	0.9877
8	1	3	6	2	0	9	1	3	2	1	3	0	0.9583	-0.5072	0.5325	0.9874
8	1	3	6	3	0	8	1	2	2	0			0.9610	-0.5153	0.5407	0.9877
8	1	3	6	4	0	7	1	3	2	1	3	0	0.9886	-0.5440	0.5693	0.9889
8	1	3	6	8	0	7	1	5	2	1	3	0	0.9952	-0.5451	0.5706	0.9888
8	1	3	7	1	0	8	1	4	2	0			0.9792	-0.5398	0.5652	0.9887
8	1	3	7	2	0	9	1	3	2	1	3	0	0.9576	-0.5122	0.5375	0.9876
8	1	3	7	3	0	10	1	2	2	0			0.9484	-0.5031	0.5283	0.9873
8	1	3	7	4	0	9	1	3	2	1	3	0	0.9654	-0.5134	0.5386	0.9878
8	1	3	7	5	0	8	1	4	2	2	3	0	0.9915	-0.5417	0.5668	0.9890
8	1	3	8	2	0	7	1	3	2	1	3	0	0.9653	-0.5404	0.5657	0.9887
8	1	3	8	3	0	8	1	2	2	0			0.9478	-0.5134	0.5388	0.9876
8	1	3	8	4	0	9	1	3	2	1	3	0	0.9530	-0.5064	0.5315	0.9875
8	1	3	8	5	0	8	1	4	2	1	3	0	0.9688	-0.5167	0.5418	0.9879
8	1	4	1	4	0	9	1	3	2	0			0.9327	-0.4891	0.5147	0.9862
8	1	4	1	5	0	10	1	4	2	0			0.9355	-0.4823	0.5077	0.9860
8	1	4	1	6	0	9	1	3	2	0			0.9471	-0.4912	0.5167	0.9865
8	1	4	2	5	0	8	1	4	2	1	3	0	0.9509	-0.4989	0.5243	0.9868
8	1	4	2	6	0	9	1	5	2	1	3	0	0.9519	-0.4917	0.5172	0.9865
8	1	4	2	7	0	8	1	4	2	1	3	0	0.9592	-0.5001	0.5257	0.9868
8	1	4	3	6	0	7	1	4	2	1	3	0	0.9665	-0.5129	0.5385	0.9873
8	1	4	3	7	0	8	1	4	2	1	3	0	0.9622	-0.5045	0.5302	0.9869
8	1	4	3	8	0	7	1	4	2	1	3	0	0.9662	-0.5130	0.5388	0.9871
8	1	4	4	1	0	6	1	3	2	0			0.9481	-0.5258	0.5519	0.9875
8	1	4	4	7	0	6	1	3	2	0			0.9535	-0.5259	0.5518	0.9876
8	1	4	4	8	0	7	1	3	2	0			0.9453	-0.5165	0.5426	0.9871
8	1	4	5	1	0	8	1	4	2	0			0.9631	-0.5050	0.5310	0.9867
8	1	4	5	8	0	7	1	4	2	1	3	0	0.9730	-0.5139	0.5398	0.9871
8	1	4	6	1	0	8	1	5	2	0			0.9663	-0.5015	0.5274	0.9866
8	1	4	6	2	0	9	1	5	2	1	3	0	0.9576	-0.4927	0.5183	0.9863
8	1	4	7	2	0	9	1	5	2	1	3	0	0.9548	-0.4925	0.5179	0.9865
8	1	4	7	3	0	10	1	4	2	1	3	0	0.9454	-0.4838	0.5092	0.9861

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c	C_1	C_2	C_3	R^2
8	1 4 8 3	0 10	1 4	2 1	3 0	0.9415	-0.4878	0.5132	0.9864
8	1 4 8 4	0 11	1 3	2 0		0.9329	-0.4795	0.5049	0.9859
8	1 4 8 5	0 10	1 4	2 1	3 0	0.9460	-0.4885	0.5138	0.9865
8	1 5 1 5	0 12	1 4	2 0		0.9228	-0.4712	0.4967	0.9854
8	1 5 2 6	0 11	1 5	2 1	3 0	0.9359	-0.4754	0.5007	0.9857
8	1 5 3 7	0 10	1 6	2 2	3 0	0.9510	-0.4845	0.5100	0.9861
8	1 5 4 8	0 9	1 5	2 1	3 0	0.9574	-0.4964	0.5222	0.9864
8	1 5 5 1	0 8	1 4	2 0		0.9391	-0.5079	0.5340	0.9866

8-Unit Patterns - Sorted by Strength

N	Pattern	F_m F_c	F_m F_c	F_m F_c	F_m F_c	C_1	C_2	C_3	R^2
8	1 5 1 5	0 12	1 4	2 0		0.9228	-0.4712	0.4967	0.9854
8	1 5 2 6	0 11	1 5	2 1	3 0	0.9359	-0.4754	0.5007	0.9857
8	1 4 8 4	0 11	1 3	2 0		0.9329	-0.4795	0.5049	0.9859
8	1 5 3 7	0 10	1 6	2 2	3 0	0.9510	-0.4845	0.5100	0.9861
8	1 4 7 3	0 10	1 4	2 1	3 0	0.9454	-0.4838	0.5092	0.9861
8	1 4 8 3	0 10	1 4	2 1	3 0	0.9415	-0.4878	0.5132	0.9864
8	1 4 8 5	0 10	1 4	2 1	3 0	0.9460	-0.4885	0.5138	0.9865
8	1 4 1 5	0 10	1 4	2 0		0.9355	-0.4823	0.5077	0.9860
8	1 3 7 3	0 10	1 2	2 0		0.9484	-0.5031	0.5283	0.9873
8	1 4 2 6	0 9	1 5	2 1	3 0	0.9519	-0.4917	0.5172	0.9865
8	1 4 6 2	0 9	1 5	2 1	3 0	0.9576	-0.4927	0.5183	0.9863
8	1 4 7 2	0 9	1 5	2 1	3 0	0.9548	-0.4925	0.5179	0.9865
8	1 5 4 8	0 9	1 5	2 1	3 0	0.9574	-0.4964	0.5222	0.9864
8	1 3 6 2	0 9	1 3	2 1	3 0	0.9583	-0.5072	0.5325	0.9874
8	1 3 7 2	0 9	1 3	2 1	3 0	0.9576	-0.5122	0.5375	0.9876
8	1 3 7 4	0 9	1 3	2 1	3 0	0.9654	-0.5134	0.5386	0.9878
8	1 3 8 4	0 9	1 3	2 1	3 0	0.9530	-0.5064	0.5315	0.9875
8	1 4 1 4	0 9	1 3	2 0		0.9327	-0.4891	0.5147	0.9862
8	1 4 1 6	0 9	1 3	2 0		0.9471	-0.4912	0.5167	0.9865
8	1 2 6 2	0 9	1 1	2 0		0.9401	-0.5382	0.5621	0.9898
8	1 4 6 1	0 8	1 5	2 0		0.9663	-0.5015	0.5274	0.9866
8	1 3 7 5	0 8	1 4	2 2	3 0	0.9915	-0.5417	0.5668	0.9890
8	1 3 8 5	0 8	1 4	2 1	3 0	0.9688	-0.5167	0.5418	0.9879
8	1 4 2 5	0 8	1 4	2 1	3 0	0.9509	-0.4989	0.5243	0.9868
8	1 4 2 7	0 8	1 4	2 1	3 0	0.9592	-0.5001	0.5257	0.9868
8	1 4 3 7	0 8	1 4	2 1	3 0	0.9622	-0.5045	0.5302	0.9869
8	1 3 1 5	0 8	1 4	2 0		0.9594	-0.5152	0.5405	0.9877
8	1 3 5 1	0 8	1 4	2 0		0.9708	-0.5171	0.5427	0.9876
8	1 3 6 1	0 8	1 4	2 0		0.9710	-0.5172	0.5427	0.9877
8	1 3 7 1	0 8	1 4	2 0		0.9792	-0.5398	0.5652	0.9887
8	1 4 5 1	0 8	1 4	2 0		0.9631	-0.5050	0.5310	0.9867
8	1 5 5 1	0 8	1 4	2 0		0.9391	-0.5079	0.5340	0.9866
8	1 2 6 3	0 8	1 2	2 1	3 0	0.9627	-0.5510	0.5749	0.9902
8	1 2 7 3	0 8	1 2	2 1	3 0	0.9514	-0.5432	0.5668	0.9901
8	1 2 5 1	0 8	1 2	2 0		0.9512	-0.5433	0.5673	0.9899
8	1 2 6 1	0 8	1 2	2 0		0.9544	-0.5498	0.5738	0.9901
8	1 3 6 3	0 8	1 2	2 0		0.9610	-0.5153	0.5407	0.9877
8	1 3 8 3	0 8	1 2	2 0		0.9478	-0.5134	0.5388	0.9876
8	1 3 6 8	0 7	1 5	2 1	3 0	0.9952	-0.5451	0.5706	0.9888
8	1 3 5 8	0 7	1 4	2 1	3 0	0.9850	-0.5270	0.5526	0.9879
8	1 4 3 6	0 7	1 4	2 1	3 0	0.9665	-0.5129	0.5385	0.9873
8	1 4 3 8	0 7	1 4	2 1	3 0	0.9662	-0.5130	0.5388	0.9871
8	1 4 5 8	0 7	1 4	2 1	3 0	0.9730	-0.5139	0.5398	0.9871
8	1 2 4 8	0 7	1 3	2 1	3 0	0.9688	-0.5553	0.5792	0.9903

N	Pattern	F_m	F_c	F_m	F_c	F_m	F_c	F_m	F_c	C_1	C_2	C_3	R^2
8	1 2 5 8	0	7	1	3	2	1	3	0	0.9730	-0.5560	0.5801	0.9903
8	1 2 6 4	0	7	1	3	2	1	3	0	1.0003	-0.5863	0.6104	0.9911
8	1 2 6 8	0	7	1	3	2	1	3	0	0.9897	-0.5845	0.6087	0.9910
8	1 2 7 4	0	7	1	3	2	1	3	0	0.9746	-0.5564	0.5802	0.9905
8	1 2 8 4	0	7	1	3	2	1	3	0	0.9620	-0.5541	0.5778	0.9905
8	1 3 2 6	0	7	1	3	2	1	3	0	0.9719	-0.5294	0.5548	0.9883
8	1 3 4 8	0	7	1	3	2	1	3	0	0.9787	-0.5305	0.5562	0.9881
8	1 3 5 2	0	7	1	3	2	1	3	0	0.9706	-0.5247	0.5503	0.9879
8	1 3 6 4	0	7	1	3	2	1	3	0	0.9886	-0.5440	0.5693	0.9889
8	1 3 8 2	0	7	1	3	2	1	3	0	0.9653	-0.5404	0.5657	0.9887
8	1 3 1 4	0	7	1	3	2	0			0.9558	-0.5227	0.5481	0.9879
8	1 3 1 6	0	7	1	3	2	0			0.9695	-0.5247	0.5502	0.9880
8	1 4 4 8	0	7	1	3	2	0			0.9453	-0.5165	0.5426	0.9871
8	1 2 5 2	0	7	1	1	2	0			0.9515	-0.5522	0.5763	0.9901
8	1 2 7 2	0	7	1	1	2	0			0.9437	-0.5511	0.5752	0.9901
8	1 2 5 7	0	6	1	4	2	1	3	0	1.0079	-0.5910	0.6154	0.9911
8	1 2 6 5	0	6	1	4	2	1	3	0	1.0292	-0.6456	0.6691	0.9929
8	1 2 6 7	0	6	1	4	2	1	3	0	1.0210	-0.6441	0.6677	0.9928
8	1 2 7 5	0	6	1	4	2	1	3	0	1.0055	-0.5908	0.6149	0.9912
8	1 3 5 7	0	6	1	4	2	2	3	0	1.0092	-0.5558	0.5815	0.9891
8	1 2 4 7	0	6	1	3	2	1	3	0	0.9893	-0.5681	0.5922	0.9906
8	1 2 8 5	0	6	1	3	2	1	3	0	0.9785	-0.5664	0.5903	0.9907
8	1 3 2 5	0	6	1	3	2	1	3	0	0.9747	-0.5383	0.5637	0.9886
8	1 3 2 7	0	6	1	3	2	1	3	0	0.9771	-0.5386	0.5642	0.9885
8	1 3 4 7	0	6	1	3	2	1	3	0	0.9918	-0.5408	0.5665	0.9885
8	1 3 4 1	0	6	1	3	2	0			0.9782	-0.5394	0.5652	0.9883
8	1 4 4 1	0	6	1	3	2	0			0.9481	-0.5258	0.5519	0.9875
8	1 4 4 7	0	6	1	3	2	0			0.9535	-0.5259	0.5518	0.9876
8	1 2 3 7	0	6	1	2	2	1	3	0	0.9771	-0.5715	0.5953	0.9909
8	1 2 5 3	0	6	1	2	2	1	3	0	0.9885	-0.5876	0.6118	0.9911
8	1 2 8 3	0	6	1	2	2	1	3	0	0.9544	-0.5622	0.5861	0.9906
8	1 2 1 5	0	6	1	2	2	0			0.9620	-0.5692	0.5929	0.9909
8	1 2 4 1	0	6	1	2	2	0			0.9628	-0.5640	0.5882	0.9904
8	1 2 7 1	0	6	1	2	2	0			0.9705	-0.5848	0.6090	0.9910
8	1 3 1 3	0	6	1	2	2	0			0.9591	-0.5489	0.5744	0.9889
8	1 3 1 7	0	6	1	2	2	0			0.9835	-0.5519	0.5775	0.9890
8	1 3 3 7	0	6	1	2	2	0			0.9580	-0.5435	0.5690	0.9887
8	1 2 5 6	0	5	1	4	2	1	3	0	1.0366	-0.6511	0.6748	0.9928
8	1 2 4 6	0	5	1	3	2	1	3	0	1.0205	-0.6036	0.6281	0.9914
8	1 2 5 4	0	5	1	3	2	1	3	0	1.0251	-0.6492	0.6727	0.9929
8	1 2 7 8	0	5	1	3	2	1	3	0	1.0002	-0.6449	0.6683	0.9929
8	1 2 8 6	0	5	1	3	2	1	3	0	1.0020	-0.6006	0.6249	0.9914
8	1 3 2 4	0	5	1	3	2	1	3	0	0.9817	-0.5662	0.5915	0.9897
8	1 3 2 8	0	5	1	3	2	1	3	0	0.9890	-0.5675	0.5931	0.9895
8	1 3 4 6	0	5	1	3	2	1	3	0	1.0134	-0.5709	0.5965	0.9896
8	1 2 3 6	0	5	1	2	2	1	3	0	0.9928	-0.5842	0.6081	0.9912
8	1 2 3 8	0	5	1	2	2	1	3	0	0.9737	-0.5813	0.6053	0.9911
8	1 2 1 4	0	5	1	2	2	0			0.9621	-0.5795	0.6033	0.9911
8	1 2 1 6	0	5	1	2	2	0			0.9711	-0.5807	0.6047	0.9911
8	1 3 3 6	0	5	1	2	2	0			0.9667	-0.5538	0.5793	0.9891
8	1 3 3 8	0	5	1	2	2	0			0.9595	-0.5533	0.5790	0.9889
8	1 2 2 6	0	5	1	1	2	0			0.9434	-0.5854	0.6089	0.9915
8	1 2 4 2	0	5	1	1	2	0			0.9768	-0.5964	0.6208	0.9911
8	1 2 8 2	0	5	1	1	2	0			0.9589	-0.5939	0.6182	0.9912
8	1 2 4 5	0	4	1	3	2	1	3	0	1.0442	-0.6658	0.6895	0.9932

N	Pattern				F_m	F_c	F_m	F_c	F_m	F_c	F_m	F_c	C_1	C_2	C_3	R^2
8	1	2	3	5	0	4	1	2	2	1	3	0	1.0186	-0.6211	0.6452	0.9920
8	1	2	4	3	0	4	1	2	2	1	3	0	1.0120	-0.6604	0.6838	0.9932
8	1	2	1	3	0	4	1	2	2	0			0.9684	-0.6135	0.6375	0.9919
8	1	2	1	7	0	4	1	2	2	0			0.9880	-0.6162	0.6404	0.9918
8	1	2	3	1	0	4	1	2	2	0			0.9829	-0.6163	0.6406	0.9918
8	1	2	8	1	0	4	1	2	2	0			0.9763	-0.6545	0.6779	0.9931
8	1	3	3	1	0	4	1	2	2	0			0.9695	-0.5849	0.6105	0.9901
8	1	3	3	5	0	4	1	2	2	0			0.9808	-0.5851	0.6103	0.9903
8	1	2	2	5	0	4	1	1	2	0			0.9519	-0.5979	0.6214	0.9918
8	1	2	2	7	0	4	1	1	2	0			0.9461	-0.5973	0.6209	0.9917
8	1	2	3	4	0	3	1	2	2	1	3	0	1.0333	-0.6872	0.7102	0.9938
8	1	2	1	2	0	3	1	1	2	0			0.9587	-0.6760	0.6991	0.9936
8	1	2	1	8	0	3	1	1	2	0			0.9885	-0.6799	0.7031	0.9936
8	1	2	2	4	0	3	1	1	2	0			0.9677	-0.6368	0.6603	0.9927
8	1	2	2	8	0	3	1	1	2	0			0.9575	-0.6361	0.6598	0.9926
8	1	2	2	1	0	2	1	1	2	0			0.9485	-0.7076	0.7297	0.9945
8	1	2	2	3	0	2	1	1	2	0			0.9596	-0.7073	0.7292	0.9946

Appendix F - Draft Design Guide

The Town Lattice Truss

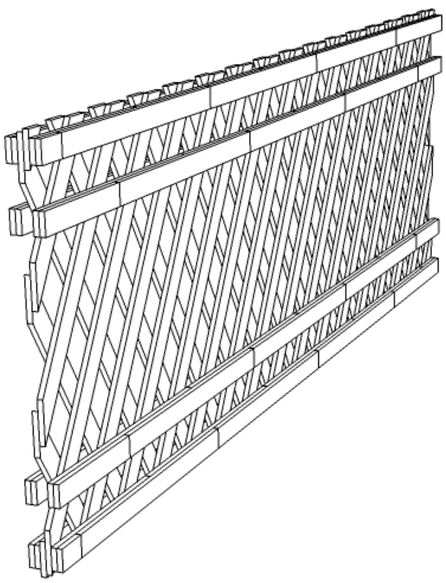
The Town Lattice Truss is an entirely wooden truss that can be fully manufactured using local labour. The truss is competitive with other bridge systems for timber-rich rural areas. The Town Lattice Truss is most appropriate in rural areas that have access to local wood and local labour and a need for spans that significantly exceed what is possible with a simple beam bridge. The truss is recommended to be used in the style of a covered bridge, which greatly increases the structures longevity and sustainability without the need for chemical preservatives.

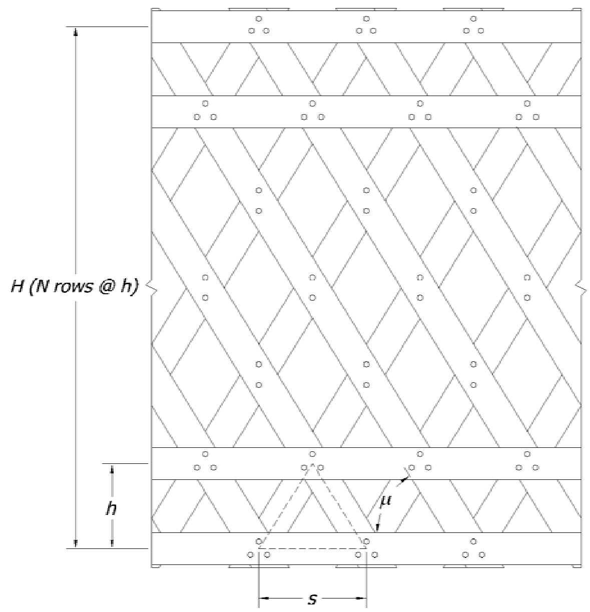
This guide provides description of the overall layout of the truss, component sizes and fabrication, truss fabrication, and bridge erection. Geometric design values are provided for a variety of spans and maximum member lengths. All values are based on a full pedestrian loading and AASHTO H10-44 vehicular loading.

Truss Details

Overall Layout

The truss is constructed from a lattice web sandwiched between longitudinal chord members, 2 on each side.





Geometry

Joint Spacing:	s	=	48”
Number of rows of joints:	N	from	6 to 8
Web angle:	μ	from	48° to 53°
Truss height:	H	from	150” to 200”

N and μ are the main variables in the design process. These variables are adjusted to give the overall height, H, needed to meet the required moment capacity.

The range of heights given assumes the use of through-trusses, which are connected overhead by a roof system. The Town Lattice Truss could also be used in smaller form with a pony truss style, or with trusses supporting the deck from underneath. The design of these scales of trusses is not addressed in this guide.

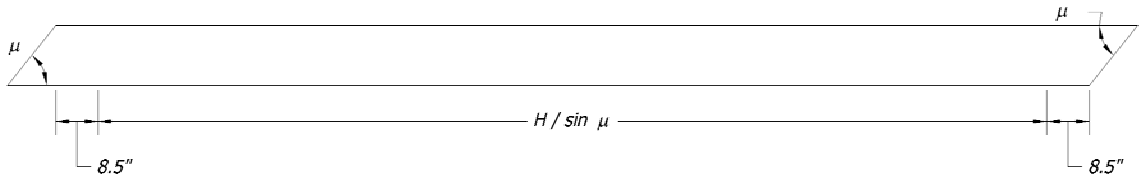
Components

<u>Members</u>

All members are fabricated with a cross-section of 3” wide by 12” deep. All web members and all chord members are identical except for those that must be shortened at the ends of the truss. Vertical end posts can be added to allow connections at the truss terminations.

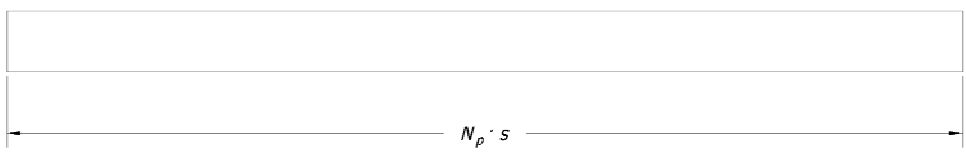
Web Members

Web members must be long enough to reach the full truss height at a given web angle, and have extra length to clear the outermost chords. Members are completed with an angled cut that creates a horizontal surface in the final truss.



Chord Members

Chord members must be an integer number (N_p) of joint spacings (s) long. This ensures all terminations occur halfway between chord connection points. The length should be the maximum possible based on the available timber.

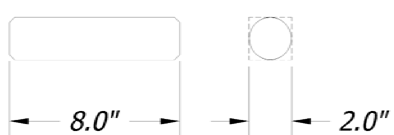


<u>Pegs</u>

All connections within the truss are made with the use of large wooden pegs (or ‘trunnels’). Pegs are typically fabricated from 2” square sections of the appropriate length, which are shaped into round pegs. Pegs should have length that is 2” greater than the total width of the section that is being connected, allowing 1” to extend beyond the members at each end.

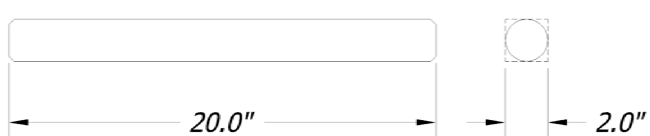
Web peg

Web pegs must have a length of 8” in order to connect two 3” web layers and have 1” clearance at each end.



Chord peg

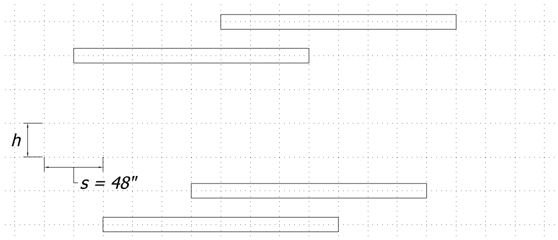
Web pegs must have a length of 20” in order to connect two 3” web layers and four 3” chord layers and have 1” clearance at each end.



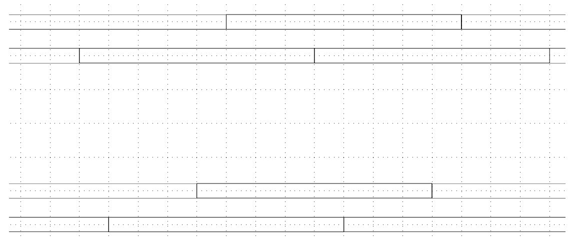
Truss Fabrication

Member Arrangement

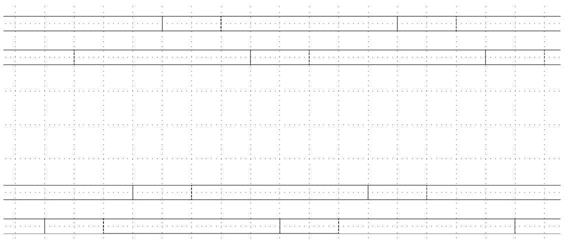
General procedure is to lay all members out in the appropriate arrangement before connecting members together. Chord members should be laid out in a way to yield the required chord termination pattern. The pattern should be staggered between adjacent chords. The steps in fabrication are illustrated below.



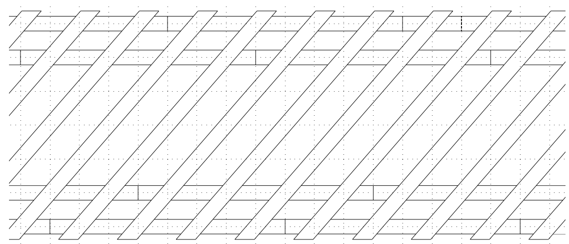
(1) Stagger initial chord members



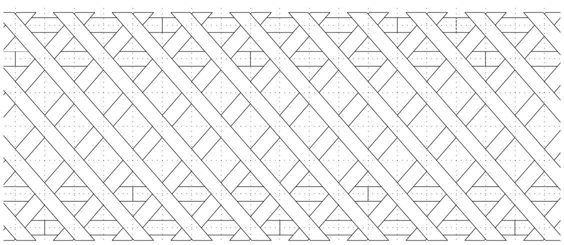
(2) Complete first chord layer



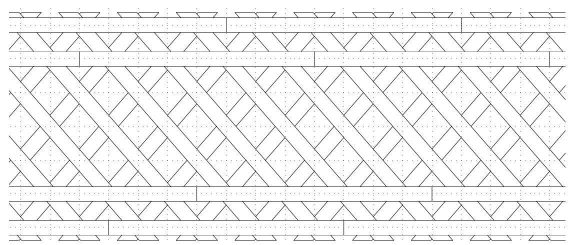
(3) Add second chord layer



(4) Add first web layer



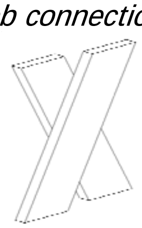
(5) Add second web layer



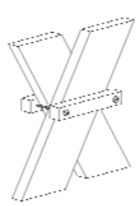
(6) Add upper two layers of chord

Pegged Connections

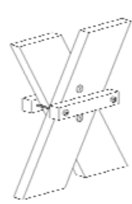
Web connection



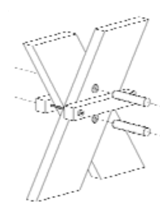
(1) Arrange members



(2) Clamp members

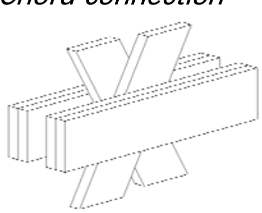


(3) Auger holes

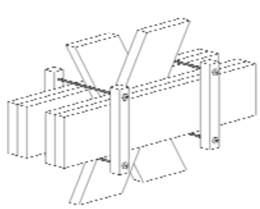


(4) Insert pegs

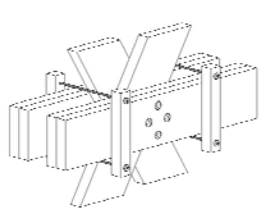
Chord connection



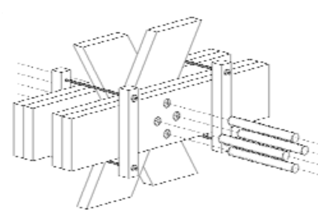
(1) Arrange members



(2) Clamp members



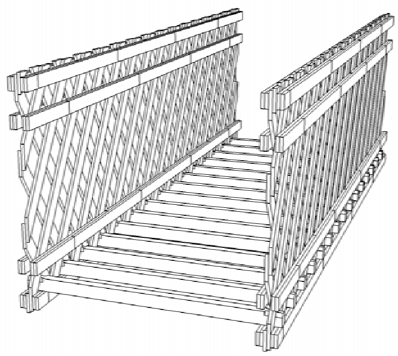
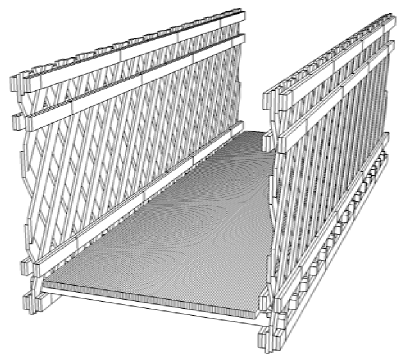
(3) Auger holes



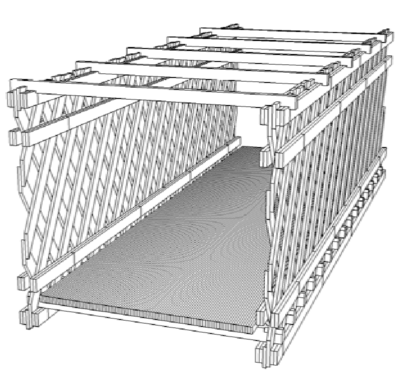
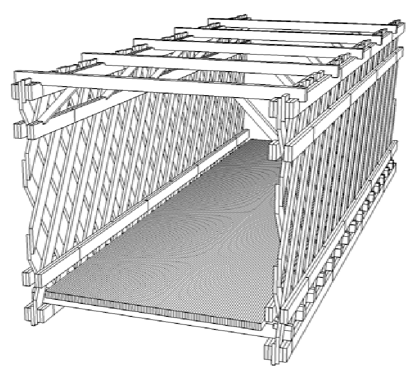
(4) Insert pegs

Bridge Fabrication

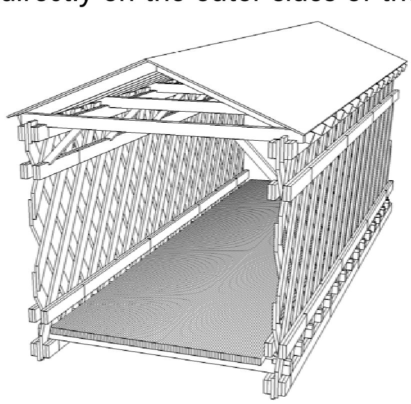
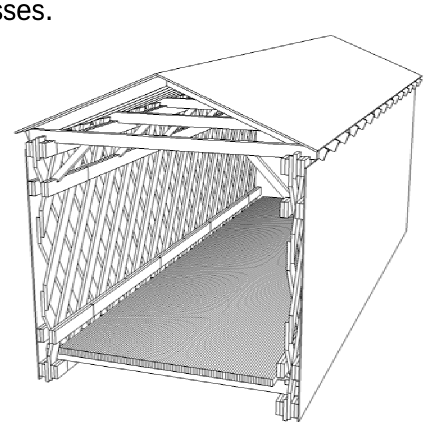
Two identical trusses are combined to form a bridge with trusses running on either side of traffic. To support traffic loading, transverse deck beams are spaced at one full joint spacing, *s*, and sit directly on top of the bottommost chord. Deck beams are overlain by a longitudinal nail-laminated deck which transfers load between deck beams and acts as a running surface.

The topmost chords of the two trusses are connected together with cross beams, which act as part of a lateral bracing system. Lateral bracing is provided with either knee braces or a complete triangular frame. Lateral bracing is connected to both the lower top chord of each truss and the cross beam.

Finally, an outer covering is provided for the entire structure to protect the structural members from moisture and deterioration. A roof structure can be connected to the topmost chords of the trusses and the lateral bracing frame if present. Siding can be mounted directly on the outer sides of the trusses.

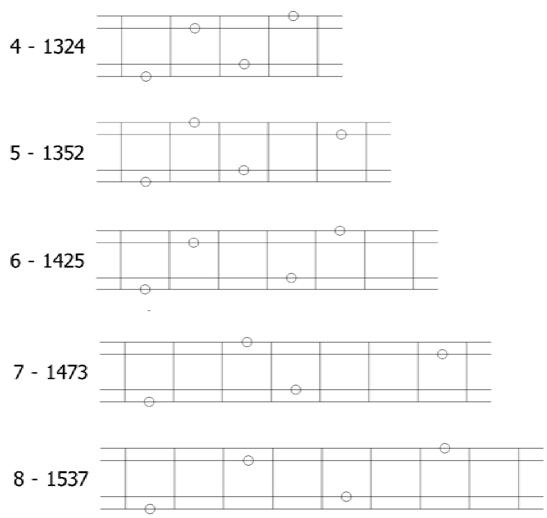
Design

Wood

The geometric design values presented in this guide are based on the use of Group B wood, as defined in Overseas Road Note 9: A Design Manual for Small Bridges (2000) from the Transport Research Laboratory of the UK Department for International Development. Group B woods are designated as lighter hardwoods having a specific gravity (SG) of less than 0.65 at 18% moisture content. A heavy hardwood (SG > 0.65) can also be used, as it will have greater strength.

Pattern

Different member lengths result in a different selection of possible patterns. For group B wood with 4-peg chord connections, the best patterns are:



Design Table

Final geometric properties shall be selected from the table below. The designer should decide the span to be crossed and the length of chord member to be used. The resulting properties are based on full pedestrian loading and AASHTO H10-44 vehicular loading.

	$N_p = 4$			$N_p = 5$			$N_p = 6$			$N_p = 7$			$N_p = 8$		
Span (ft)	H (ft)	N	μ (deg)	H (ft)	N	μ (deg)	H (ft)	N	μ (deg)	H (ft)	N	μ (deg)	H (ft)	N	μ (deg)
75	13.8	7	49	-	-	-	-	-	-	-	-	-	-	-	-
80	15.9	7	53	13.8	7	49	-	-	-	-	-	-	-	-	-
85	-	-	-	15.9	7	53	14.3	7	50	14.3	7	50	-	-	-
90	-	-	-	-	-	-	15.9	7	53	15.9	7	53	13.8	7	49
95	-	-	-	-	-	-	-	-	-	-	-	-	15.4	7	52
100	-	-	-	-	-	-	-	-	-	-	-	-	16.7	8	50